33 CFR 328.3 notes "other high tides that occur with periodic frequency." These "other high tides" might be locally interpreted to include Mean High Water (MHW), Mean Higher High water (MHHW), or perhaps Highest Astronomical Tide (HAT). MHHW is below MHWS. HAT would represent the maximum tidal heights exclusive of storm surges. Local usage will determine which of these tidal datums should be used to define a permit limit on the shoreline. Regardless of the reference datum, it should be based on a firm mathematical computation from gage data, as illustrated for Spring High Water Tides (SHWT) below.

(2) It should be noted that MHW and MHHW datums are determined from all observed tides, including storm surge or other weather effects—the 33 CFR 328.3 definition excludes storm surge tides. By using mean values over a 19-year period, the effect of tides biased by weather effects (both extreme highs and extreme lows) are averaged out.

(3) Highest Astronomical Tide (HAT) is not based on observed gage data but is defined as the individual highest and lowest predicted tides over the NTDE 1983-2001 time period using the NOAA tide prediction procedures. NOAA predicted tides, including HAT, do not include daily and weekly weather effects in their elevations, but do include annual and semi-annual constituents that are driven by average seasonal changes in mean sea level.

b. Spring High Water Tides. Spring tides of increased range occur semimonthly as the result of the Moon being new or full. The spring range of tide is the average range occurring at the time of spring tides and is most conveniently computed from the harmonic constants. It is larger than the mean tide range (Mn) where the type of tide is either semi diurnal or mixed, and is of no practical significance where the type of tide is predominantly diurnal. The average height of the high waters of the spring tides is called Spring High Water Tides (SHWT) or Mean High Water Springs (MHWS), and the average height of the corresponding low waters is called Spring Low Water or Mean Low Water Springs (MLWS). Reference "*Tidal Datums and Their Applications*" (NOAA 2001).

(1) Historically, the international community, when it used Mean Low Water Springs (MLWS) as a chart datum, derives it by MLWS = Z0 - (M2 + S2), or the sum of the amplitudes of the semidiurnal (Z0) and solar M2 and S2 harmonic constituents below a mean value. Likewise, MHWS = Z0 + (M2 + S2). So high water spring tide datums are not based on tabulation of observations, but on harmonic analysis of observations.

(2) NOAA does not publish MHWS at its NWLON tide gage stations. However, they do publish the Spring Tide Ranges for selected prediction stations. The Spring Tide Range can be used to approximate the MHWS range by adding half the Spring Tide Range to the Mean Tide Level published for a gage station. This is only applicable for semi-diurnal tides—Spring Tide Ranges are not published for areas of diurnal tides (most of the Gulf of Mexico, etc.). In diurnal or mixed tide regions, the Highest Astronomical Tide (HAT) could be considered in lieu of MHWS because it can be determined the same way regardless of type of tide.

c. Mean High Water datum.. Mean High Water (MHW) is a reference for Section 10 boundaries— labeled as "Mean High Tide" in Figure 7-2. MHW datum is defined as the average

height of all high waters at a tide station referenced to a 19-year period—see Chapter 2. MHW datum is always below HTL.

d. Non-tidal wetland. A non-tidal wetland is defined as a wetland (i.e., a water of the United States) that is not subject to the ebb and flow of tidal waters. Non-tidal wetlands contiguous to tidal waters are located landward of the high tide line (e.g., the Spring High Tide Line).

e. Tidal wetland. A tidal wetland is a wetland (i.e., a water of the United States) that is inundated by tidal waters—reference 33 CFR 328.3. Tidal waters end where the rise and fall of the water surface can no longer be practically measured in a predictable rhythm due to masking by other waters, wind, or other effects. Tidal wetlands are located channel ward of the high tide line (i.e., spring high tide line) and are inundated by tidal waters two times per lunar month, during spring high tides.

f. Tidal datum computations. Methods for determining tidal datums using water level gage comparison techniques are covered in state and federal publications. The primary source for most tidal datum computations is in the NOAA "Computational Techniques for Tidal Datums Handbook Computational Techniques" (NOAA 2003). This reference outlines methods for establishing tidal datums from gage observational data, including simultaneous comparison methods (e.g., Range-Ratio) used to transfer tidal datums from an established gage site to a remote project (permit) site. Typically, MHW datums are transferred using these simultaneous comparison methods. This NOAA manual does not cover establishment of an "Ordinary High Water Mark" (OHWM) since an OHWM demarcation is not always based on direct gage observations.

(1) The period of simultaneous gage comparisons is dependent on the distance from the site to the NWLON gage, the mean tide range, and local or individual state requirements. Simultaneous gage comparison periods of 3, 7, to 30 days are common in CONUS. Temporary staff gages may be used for short-term comparisons.

(2) Many coastal states have statutory and regulatory requirements for defining and observing new tidal datums at a project site. One example is in Florida Statutes, Chapter 177, Part II of the "Florida Coastal Mapping Act of 1974." Where these statues are applicable, surveyors portraying tidal elevations on permit drawings must have met the minimum technical standards prescribed in the statutes. For example, the Florida Department of Environmental Protection Chapter 177 statutes are relatively rigid regarding establishment of tidal datums, as extracted below. (Other coastal states have similar statutory or regulatory requirements.)

> *177.36 Work to be performed only by authorized personnel.--The establishment of local tidal datums and the determination of the location of the mean high-water line or the mean low-water line must be performed by qualified personnel licensed by the Board of Professional Surveyors and Mappers or by representatives of the United States Government when approved by the department.*
> *177.37 Notification to department.--Any surveyor undertaking to establish a local tidal datum and to determine the location of the mean high-water line or the mean low-water*

line shall submit a copy of the results thereof to the department [Department of Environmental Protection].within 90 days after the completion of such work, if the same is to be recorded or submitted to any court or agency of state or local government.

177.38 Standards for establishment of local tidal datums.--
(1) Unless otherwise allowed by this part or regulations promulgated hereunder, a local tidal datum shall be established from a series of tide observations taken at a tide station established in accordance with procedures approved by the department. In establishing such procedures, full consideration will be given to the national standards and procedures established by the National Ocean Service [NOAA CO-OPS].
(2) Records acquired at control tide stations, which are based on mean 19-year values, comprise the basic data from which tidal datums are determined.
(3) Observations at a tide station other than a control tide station shall be reduced to mean 19-year values through comparison with simultaneous observations at the appropriate control tide stations. The observations shall be made continuously and shall extend over such period as shall be provided for in departmental regulations.
(4) When a local tidal datum has been established, it shall be preserved by referring it to tidal bench marks in the manner prescribed by the department.

g. Survey procedures. Once a computed tidal datum at a site is established on a gage reference PBM, the MHW, MLW, or MHWS demarcation line can be staked out using total stations, differential levels, or RTK/RTN methods. On critical projects, the local gage PBMs should be set at stable locations for future reference and use in construction. Surveyed horizontal relationships between demarcation lines and PBMs to property corners are also normally required. CORS/OPUS techniques may be employed if a general NAD83 mapping reference is required—see Chapter 3.

7-5. Boundary Uncertainties Due to Water Level Datum Errors. The uncertainty in the value of a water level datum (e.g., MLW, MHW, OHWL, MTL) translates into a horizontal uncertainty of the location of a marine boundary when the datum line is surveyed to the land—reference "*Tidal Datums and Their Applications*" (NOAA 2001). Table 7-2 expresses the uncertainty in the marine boundary as a function of the slope (or grade) of the land. The greatest errors in the determination of a marine boundary occur for relatively flat terrain, which is characteristic of broad sections of the Atlantic and Gulf Coasts.

Table 7-2. Error in Position of Marine Boundary as a Function of the Slope of the Land given a 0.1 ft Vertical Datum Error. (Source: NOAA 2001)

Slope %	Degree of Slope (degrees)	Horizontal Error [1] (feet)
0.1	0.05	106
0.2	0.1	49
0.5	0.3	20
1.0	0.6	10
2.0	1	5
5.0	3	2
10.0	6	1
50.0	27	0.2
100.0	45	0.1

[1] error = 0.1 ft x cot (slope in degrees)

For example, a ±0.1 ft error in transferring a tidal datum from a gage to a project site on a 2% grade will equate to a ±5 ft horizontal error of the boundary demarcation line. This ± 0.1 ft "relative" uncertainty in a tidal datum does not include the regional (or global) uncertainty of the datum at the master gage. See Chapters 4 and 9 for discussions on the absolute and relative accuracies and uncertainties of tidal datums. If the tidal epoch at the project site has not been updated to the current epoch, then the error at this 2% grade site could be (+) 14 ft—14 ft landward given a 0.25 ft apparent sea level rise between epochs. The impact of these horizontal uncertainties (or biases) may or may not be significant, depending on the nature of a dredge or fill permit.

7-6. Section 10 Authority: Geographic and Jurisdictional Limits of Oceanic and Tidal Waters. Section 10 of the Rivers and Harbors Appropriations Act of 1899 (33 U.S.C. 403) requires approval prior to the accomplishment of any work or placement of any structure in navigable waters of the United States, or which affects the course, location, or condition of such waters with respect to navigable capacity. Typical activities requiring Section 10 permits include construction of piers, wharves, bulkheads, dolphins, marinas, ramps, floats intake structures, cable or pipeline crossings, and dredging and excavation. A Section 10 permit is required for all work, including structures, seaward of the "annual high water line" (e.g., MHW) in navigable waters of the United States, defined as waters subject to the ebb and flow of the tide, as well as a few of the major rivers used to transport interstate or foreign commerce. An example of a Section 10 project is shown in Figure 7-4.

a. Ocean and coastal waters. The navigable waters of the United States over which USACE regulatory jurisdiction extends include all ocean and coastal waters within a zone three

geographic (nautical) miles seaward from the baseline (The Territorial Seas). Wider zones are recognized for special regulatory powers exercised over the outer continental shelf. (See 33 CFR 322.3(b)).

(1) Baseline defined. Generally, where the shore directly contacts the open sea, the line on the shore reached by the ordinary low tides (e.g., MLW) comprises the baseline from which the distance of three geographic miles is measured. The baseline has significance for both domestic and international law and is subject to precise definitions. Special problems arise when offshore rocks, islands, or other bodies exist, and the baseline may have to be drawn seaward of such bodies.

(2) Shoreward limit of jurisdiction. Regulatory jurisdiction in coastal areas extends to the line on the shore reached by the plane of the mean (average) high water—i.e., Mean High Water (MHW). Where precise determination of the actual location of the line becomes necessary, it must be established by survey with reference to the available tidal datum, preferably averaged over a period of 18.6 years. Less precise methods, such as observation of the "apparent shoreline" which is determined by reference to physical markings, lines of vegetation, or changes in type of vegetation, may be used only where an estimate is needed of the line reached by the MHW.
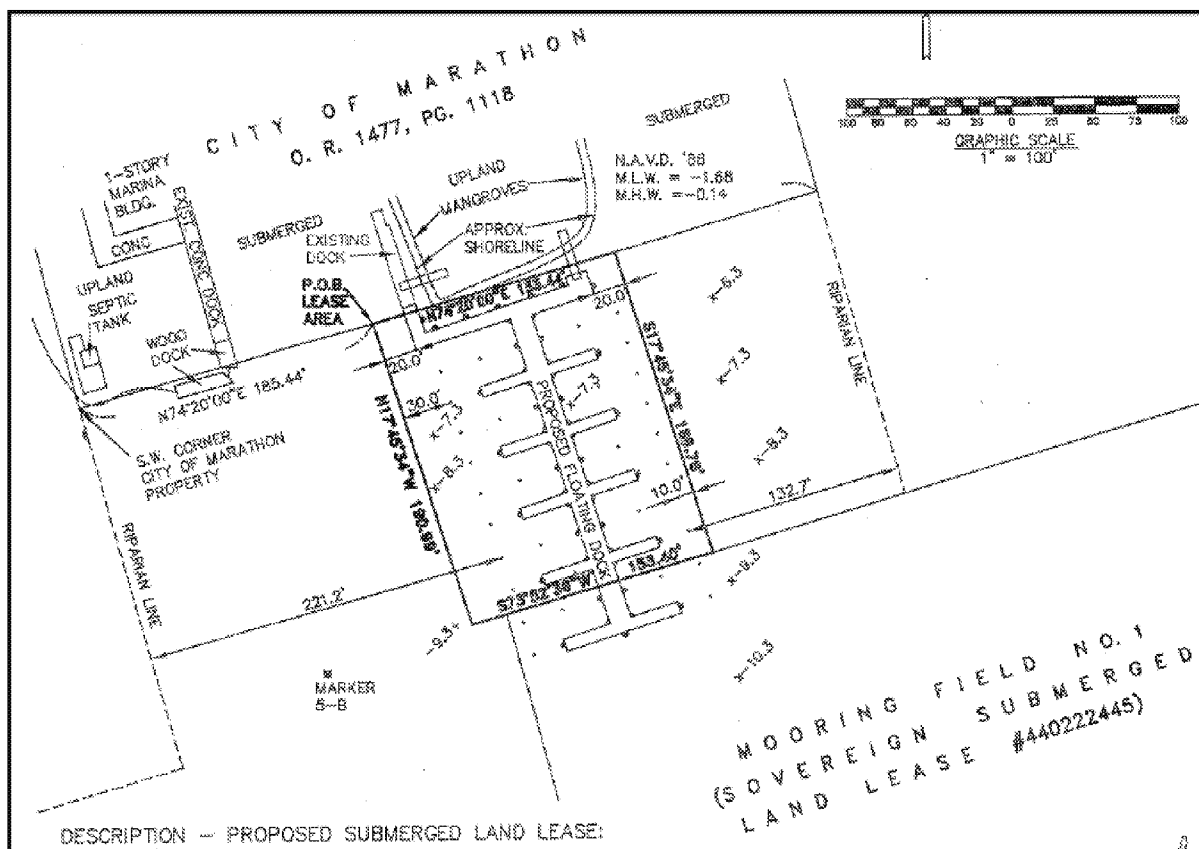

Figure 7-4. Example of Section 10 permit application with tidal datums and elevations referenced to NAVD88. (Jacksonville District)

b. Bays and estuaries. Regulatory jurisdiction extends to the entire surface and bed of all water bodies subject to tidal action. Jurisdiction thus extends to the edge of all such water bodies, even though portions of the water body may be extremely shallow, or obstructed by shoals, vegetation, or other barriers. Marshlands and similar areas are thus considered "navigable in law," but only so far as the area is subject to inundation by the mean high waters. The relevant test is therefore the presence of the mean high tidal waters, and not the general test described above, which generally applies to inland rivers and lakes.

c. Tidal datum computations. Methods for determining MHW tidal datums at remote sites using simultaneous comparison techniques were outlined in paragraph 7-4. Required tidal observation periods are highly site dependent—based on tide range and distance between the comparison gage and site.

7-7. Section 404 Authority: Limits of Jurisdiction—Dredged or Fill Material. Section 404 of the Clean Water Act of 1977 (33 U.S.C. 1344) regulates the discharge of dredged, excavated, or fill material in wetlands, streams, rivers, and other U.S. waters. Jurisdictional boundaries relative to tidal and non-tidal waters are defined below. Typical activities requiring Section 404 permits include depositing of fill or dredged material in waters of the U.S. or adjacent wetlands, site development fill for residential, commercial, or recreational developments (see Figure 7-5), construction of revetments, groins, breakwaters, levees, dams, dikes, and weirs, and placement of riprap and road fills.

a. Territorial Seas. The limit of jurisdiction in the territorial seas is measured from the baseline in a seaward direction a distance of three nautical miles. (Reference 33 CFR 329.12)

b. Tidal Waters of the United States. The landward limits of jurisdiction in tidal waters:

(1) Extends to the "high tide line" (HTL), or

(2) When adjacent non-tidal waters of the United States are present, the jurisdiction extends to the limits identified in paragraph (c) of this section.

c. Non-Tidal Waters of the United States. The limits of jurisdiction in non-tidal waters:

(1) In the absence of adjacent wetlands, the jurisdiction extends to the OHWM, or

(2) When adjacent wetlands are present, the jurisdiction extends beyond the OHWM to the limit of the adjacent wetlands.

(3) When the water of the United States consists only of wetlands, the jurisdiction extends to the limit of the wetland.
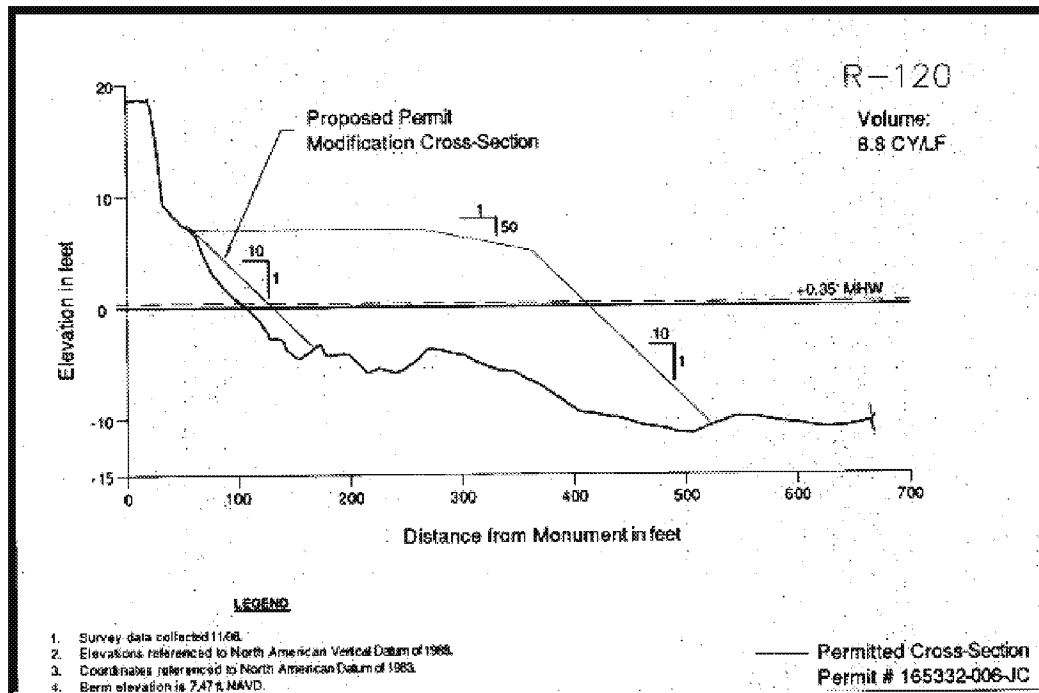
Figure 7-5. Sample Section 404 and Section 10 permit application—beach renourishment project. Cross-section and berm elevations are referenced to NAVD88. MHW relationship above NAVD88 is indicated. (The HTL is not indicated).

7-8. Section 103 Authority: Ocean Dumping of Dredged Material. Section 103 authority in the Marine Protection, Research and Sanctuaries Act (33 U.S.C. 1413), and related statutes, involves permits for ocean dumping in confined disposal sites. Vertical reference datums on these permits may be either orthometric or tidal. It is desirable that the datums be on the latest epochs and the orthometric-tidal relationship be specified. Section 103 permits can also involve a variety of site dependent parameters and restrictions. Those significant to geodetic datums involve dredge positioning/monitoring systems that record dredge/scow positions and changes in draft—e.g., the USACE "Silent Inspector" system. Currently many dredge control systems are positioned using RTK techniques. Thus, dredge draft and/or hopper drag arm elevations can be directly related to the orthometric, ellipsoidal, or tidal datum. Ocean disposal restrictions may involve both horizontal and vertical height restrictions, in addition to various turbidity and biologic criteria. Deep ocean sites (i.e., > 100 ft) may require periodic monitoring surveys to check for material dispersion.

7-9. Marine Boundaries in Coastal Areas Defined by Tidal Datums. The following material in this section is excerpted from NOAA 2001. It provides an overview of the marine boundaries defined by tidal datums in the various states.

    a. General. Chart datum, MLLW, is the elevation of the baseline for many marine boundaries, including most which are recognized by the United Nations Convention on the Law of the Sea. However, baselines may differ in position for the purposes of different statutes. The

baselines (see Figure 7-6) usually consist of points or line segments on these tidal datum lines from which the marine boundaries are measured and constructed.
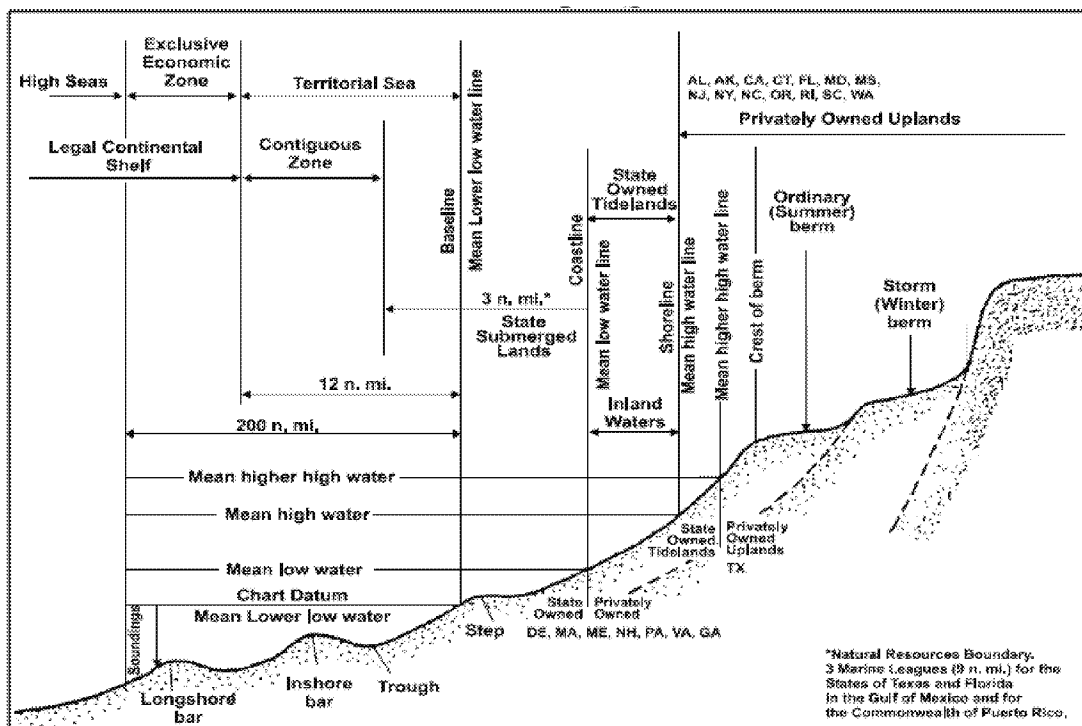

Figure 7-6. The principal tidal datums related to a beach profile. The intersection of the tidal datum with land determines the landward edge of a marine boundary.

b. Marine boundaries. As delineated in Figure 7-6, the marine boundaries of the U.S. are:

(1) Private U.S. property exists in most cases landward of MHW.

(2) State-owned tidelands exist between MHW and MLW in most cases. Refer to Figure 7-6 for individual cases. U.S. Inland Waters are concurrently defined to exist between MHW and MLW for the purpose of marine navigation.

(3) A state's "Submerged Lands Boundary" extends seaward three nautical miles from MLW, except for Texas and the Gulf coast of Florida where it terminates at nine nautical miles. In this band, plus the state-owned tidelands, the states exercise the "Public Trust Doctrine," subject to federal supremacy.

(4) The "Territorial Sea Boundary" extends 12 nautical miles seaward of MLLW. It is also known as the Marginal Sea, Marine Belt, Maritime Belt, 12-Mile Limit, and Adjacent Sea Boundary. Historically, this boundary was three nautical miles; it was changed to its present 12-mile limit in 1988. In the Territorial Sea, the sovereignty of the nation extends to the airspace above, the subsoil, the water, and the resources.

(5) The "Contiguous Zone Boundary" occurs at 12 nautical miles from MLLW. In the U.S., the Territorial Sea and Contiguous Zone are coterminous. In the contiguous zone, the nation may exercise rights to protect its interests, but does not exert sovereign control. The main purpose of the contiguous zone is to exert control over shipping near a nation's coast. Under the United Nations Convention on the Law of the Sea, a coastal nation may declare a Contiguous Zone between 12 and 24 nautical miles.

(6) The 200-mile "Fishery Conservation Zone" extends seaward from MLLW.

(7) The Presidential Proclamation 5030 of March 1983, established the "Exclusive Economic Zone" (EEZ), which claimed rights to living and mineral resources and jurisdiction of approximately 3.9 billion acres. The baseline for demarcation of the EEZ is the MLLW boundary of the Territorial Sea and extends 200 nautical miles. It should be noted that different coastal nations have different definitions of their ordinary low water. These definitions are not usually consistent with NOS definitions.

c. Mean High Water Line. The Mean High Water Line (MHWL) is the coastal boundary between private and state property with the following exceptions:

(1) Maine, New Hampshire, Massachusetts, Pennsylvania, Delaware, Virginia, and Georgia use the Mean Low Water Line (MLWL).

(2) Texas uses the Mean Higher High Water Line (MHHWL) when Spanish or Mexican grants are involved.

(3) Louisiana has adopted the civil law boundary of the line of highest winter tide.

(4) In Hawaii, the upland owner has title to the upper reaches of the wash of the waves.

d. Demarcation of MHWL. In order to map tidal boundaries such as MHWL or MLWL, and determine the latitude and longitude coordinates of their intersection with the coast, the surveyor performs the following basic procedures:

(1) Obtain the published bench mark information at or near the location.

(2) Find the tidal bench marks and run a closed line or loop of differential levels from the bench marks to that part of the shore where the boundary is to be located, run levels along the shoreline, and mark or stake points at intervals along the shore in such a manner that the ground at each point is at the elevation of the tidal datum.

(3) If the boundary is to be mapped, the horizontal distances and directions, or bearings, between each of these points and between those points or features in the area, and between the points and the horizontal control stations are measured so that the boundary may be plotted on a plat or map to the exact scale ratio and in true relation to other boundaries.

7-10. <u>Permit Application Checklist</u>. Table 7-3 outlines some reference datum issues that may warrant review on a permit application.

Table 7-3. Permit Application Checklist for Issues Relating to Geospatial Datums.

Reference Bench Mark for Project Site Surveys
    Datum noted
    Elevation of bench mark noted
    NSRS PID noted, if applicable
    If legacy reference datum (e.g., NGVD29) relationship to NAVD88 identified
    If pool/reservoir/river stage, relationship to NAVD88 identified

Topographic Surveys
    Reference datum identified
    Reference bench mark identified
    Horizontal reference datum identified
    Quantity take off metadata/source noted
    Survey date & source metadata noted

Boundary Survey
    Survey conforms to state minimum technical standards, as applicable

Tidal Datum Transfers
    Primary reference gage identified
    Tidal epoch noted
    Local site gage PBM noted
    Datum transfer to project site meets state minimum technical standards, as applicable
    Datum transfer computation metadata identified

7-11. <u>Example: Sections 10 and 404 Permit Application Involving Tidal Limits</u>. The following example is excerpted from a permit application submitted to the Jacksonville District. The permit involved a bulkhead relocation and fill in Florida tidal waters. This permit is typical of applications for fill in navigable waters or wetlands below the high tide line or MHW line. Some aspects in this example are simulated since this particular permit application did not detail the specific procedures used in transferring tidal datums from an established NOAA NWLON gage.

    a. Background. The following general description of the project is excerpted from the permit application and subsequent District technical reviews and findings.

        "Project Description: The applicant proposes to construct a bulkhead 9 to 27 feet water ward in front of an existing bulkhead and fill approximately 1,660 square feet of waters of the United States with approximately 187.5 cubic yards of fill material to extend the yard.

Statutory authority: Section 10 of the Rivers and Harbors Act of 1899 and Section 404 of the Clean Water Act of 1972, as amended.

The existing project area consists of open water and inundated floodplain classified as estuarine, subtidal, unconsolidated bottom. The on-site vegetation consists of water oak (Quercus nigra), arrowhead (Sagittaria lancifolia), wild taro (Colocasia esculenta) and other emergent vegetation. The onsite vegetative communities were classified according to the Florida Department of Transportation's Florida Land Use, Cover and Forms Classification System.

The waters of the United States (wetlands) at the site consist of the tidal floodplain of Doctors Lake, a navigable water of the United States. Doctors Lake is an elongated 3,500 acre embayment situated on the west side of the St. Johns River. It is situated in the tidal, brackish part of the Lower St. Johns River Basin (LSJRB). The lake is about five (5) miles long by one (1) mile wide, connected to the St. Johns River by an approximately 0.25 mile wide opening at its northeast end. Doctors Lake has no freshwater tributaries, making tidal exchange with the St. Johns River the lake's largest source of water ... Tidal currents play an important role in determining estuarine water quality. Tidal currents that flow into Doctors Lake on flood tide and out during ebb tide dilute and transport pollutants over each tidal cycle and thereby flush the system. Tidal circulation provides the dominant flushing mechanism in the lake. The tidal prism or the volume of water exchanged during each half tidal cycle plays an important role on tidal circulation. Reduction of intertidal shoreline can reduce the total prism of water exchanged."

Figure 7-7 depicts the project location relative to the St. Johns River, which flows into the Atlantic Ocean some 30 miles downstream of the permit site. An historic NOAA tide gage station is located directly across the lake from the permit site. Tidal bench marks at this gage site have been connected to the NSRS and have reliable NAVD88 adjusted elevations.

   b. Permit site plan details. Figure 7-8 (excerpted from the original permit application) shows the plan layout of the existing bulkhead and proposed extended bulkhead into tidal waters. The "MHW" contour in depicted in plan, along with Section A-A. The section A-A view notes the elevations of MHW and HTL. The source of the elevations was noted as a tidal PBM at NOAA tide gage 872 0406 (DOCTORS LAKE, PEORIA POINT). Elevations were referenced to NGVD29. NAVD88 control is readily available—a previous permit may have been referenced to NGVD29. There was no indication of the source of the HTL elevation (1.71 ft above NGVD29). No metadata was included in the permit detailing the source of the Mean High Water contour depicted in the plan view. It is presumed this underwater contour was derived from a topographic/boundary survey of the site performed prior to the permit application.
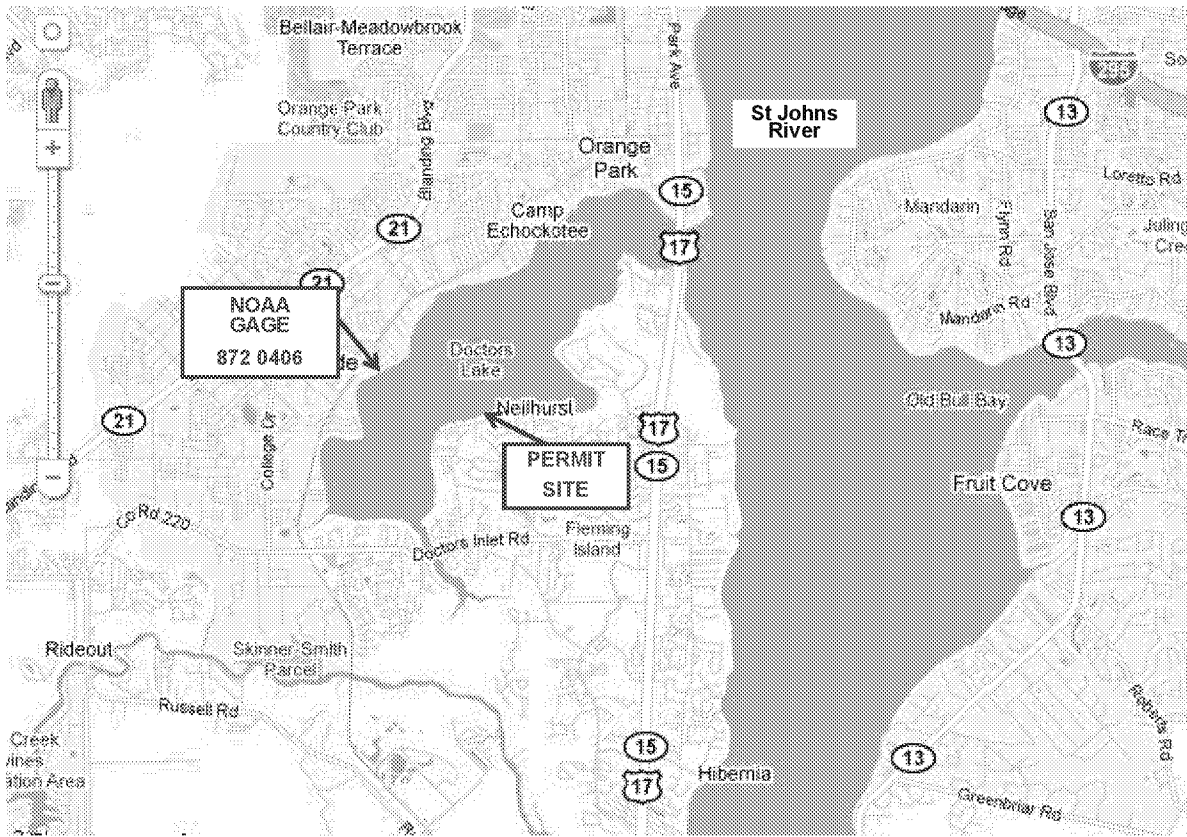
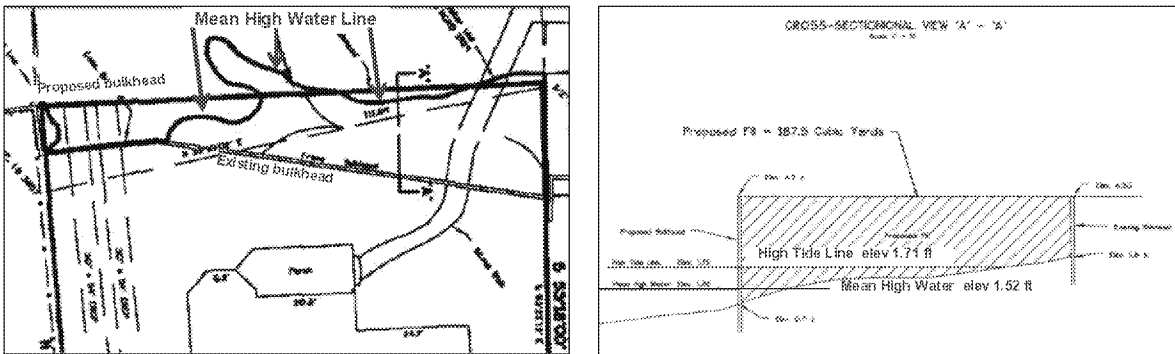Figure 7-7. Permit site and reference NOAA gage locations in tidal inlet.

Figure 7-8. Permit site: plan and section views.

Figure 7-9. Apparent surface elevations above surveyed MHW and HTL elevations.

c. NOAA datasheet for gage station 872 0406. The data in Figure 7-10 is taken from the NOAA tide gage directly across the lake from the project site. This data would be used to compare or transfer tidal datums from the gage site to the permit site, as outlined below.

```
Tidal datums at DOCTORS LAKE, PEORIA POINT based on:

     LENGTH OF SERIES:        5 MONTHS
     TIME PERIOD:             June 1978 - October 1978
     TIDAL EPOCH:             1983-2001
     CONTROL TIDE STATION:    8720496 GREEN COVE SPRINGS, ST. JOHNS R.

Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

     MEAN HIGHER HIGH WATER (MHHW)               =  0.278 [0.91 ft]
     MEAN HIGH WATER (MHW)                       =  0.257 [0.84 ft]
     MEAN TIDE LEVEL (MTL)                       =  0.136 [0.45 ft]
     MEAN SEA LEVEL (MSL)                        =  0.130
     NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)   =  0.117 [0.38 ft]
     MEAN LOW WATER (MLW)                        =  0.014
     MEAN LOWER LOW WATER (MLLW)                 =  0.000


National Geodetic Vertical Datum (NGVD 29)
Bench Mark Elevation Information          In METERS above:

     Stamping or Designation              MLLW        MHW

     0406 C 1978                          3.878       3.621
     0406 A 1978             [10.12 ft]   3.084       2.827
     0406 D 1978                          6.638       6.381
```

Figure 7-10. NOAA datasheet for tide gage 872 0496 (Doctors Lake, Peoria Point).

d. NGS datasheet for Tidal Bench Mark 872 0406A. Figure 7-11 is excerpted directly from the NGS datasheet for PBM "872 0406A." This datasheet provides reference NAVD88 relationships at the tide gage from which differential leveling or RTK surveys can be performed. This sheet also provides the relationship between NAVD88 and NGVD29, which can be transferred to the project site.

```
BC1437  TIDAL BM     -   This is a Tidal Bench Mark.
BC1437  DESIGNATION -   872 0406 A
BC1437  PID          -   BC1437
BC1437  STATE/COUNTY-   FL/CLAY
BC1437  USGS QUAD    -   MIDDLEBURG (1993)
BC1437                           *CURRENT SURVEY CONTROL
BC1437 _____
BC1437* NAD 83(1986)-  30 07 11.      (N)    081 45 33.     (W)    SCALED
BC1437* NAVD 88      -         2.973  (meters)     9.75  (feet)   ADJUSTED
BC1437 _____
BC1437  GEOID HEIGHT-       -28.13  (meters)                      GEOID09
BC1437  DYNAMIC HT  -          2.969  (meters)     9.74  (feet)   COMP
BC1437  MODELED GRAV-    979,343.2   (mgal)                       NAVD 88
BC1437  VERT ORDER   -  FIRST      CLASS II
BC1437                           SUPERSEDED SURVEY CONTROL
BC1437
BC1437  NAVD 88 (06/15/91)    2.964   (m)      9.72   (f) UNKNOWN    1 2
BC1437  NGVD 29 (09/01/92)    3.289   (m)     10.79   (f) ADJUSTED   1 2
BC1437
BC1437.Superseded values are not recommended for survey control.
BC1437.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
BC1437.See file dsdata.txt to determine how the superseded data were derived.
```

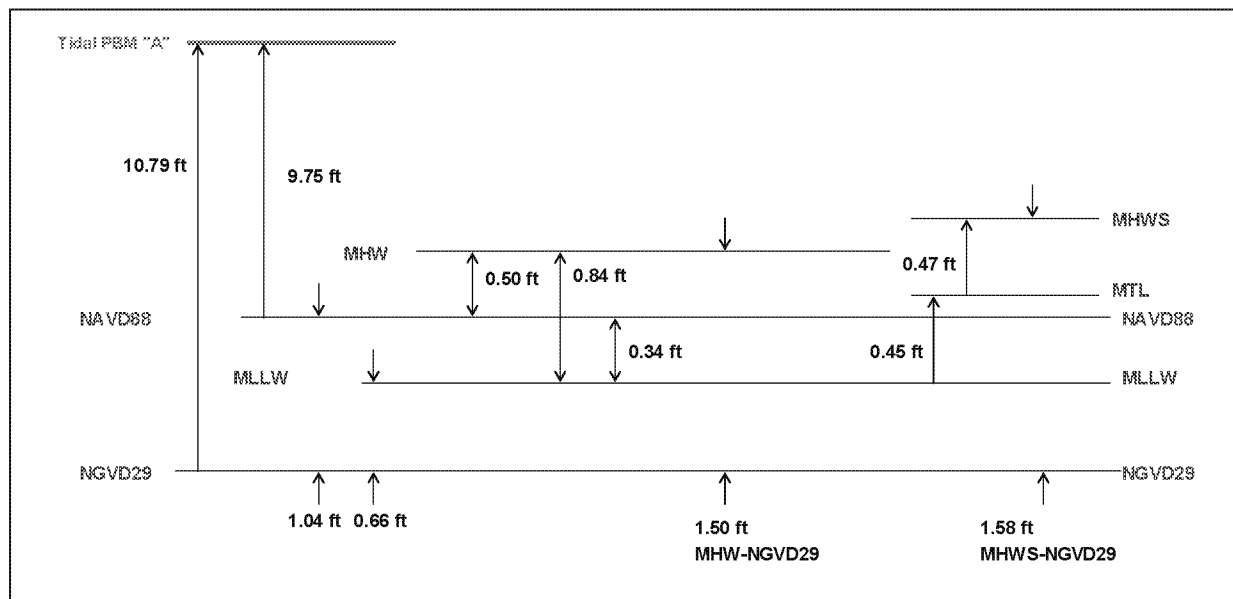Figure 7-11. NGS datasheet for tidal bench mark 872 0406A.

Figure 7-12. Datum relationships at the NOAA DOCTORS LAKE, PEORIA POINT gage.

e. Determining tidal datum relationships to orthometric datums. Using data from the NOAA CO-OPS gage site and the NGS tidal bench mark, the tidal reference datums at the permit site can be referenced to an orthometric datum (NGVD29 or NAVD88) as indicated in Figure 7-12 and the following computations.

(1) MHW elevation relative to NGVD29.

Tidal PBM "A" elevation   10.79 ft above NGVD29 (from NGS Datasheet)
Tidal PBM "A" elevation   9.75 ft above NAVD88 (from NGS Datasheet)

NAVD88 - NGVD29:	(+)	1.04 ft (concluded from above)
NAVD88 - MLLW:	(-)	0.38 ft (from CO-OPS Datasheet)
MHW - MLLW:	(+)	0.84 ft (from CO-OPS Datasheet)
MHW above NGVD29:		1.50 ft

Similarly, MHW datum can be related to NAVD88:

MHW - MLLW:	(+)	0.84 ft (from CO-OPS Datasheet)
NAVD88 - MLLW:	(-)	0.38 ft (from CO-OPS Datasheet)
MHW above NAVD88:		0.46 ft

(2) High Tide Line (HTL) determination. As stated earlier, the permit application did not indicate the source of the HTL elevation (1.71 ft above NGVD29) shown on the section view. Since the HTL is approximately related to MHWS datum, NOAA station prediction data may be used to estimate the MHWS elevation. The following data in Figure 7-13 is taken from the NOAA tidal predictions for this gage site:

FLORIDA, St. Johns River Predictions Station			Mean Range (ft)	Spring Range (ft)	Mean Tide Level (ft)
	Latitude	Longitude			
Peoria Point, Doctors Lake	30° 07.2'	81° 45.5'	0.80	0.93	0.45

Figure 7-13. NOAA tidal predictions for Doctors Lake.

(3) Estimated High Tide Line based on NOAA published Spring Tide Range relative to Mean Tide level published on the CO-OPS station Datasheet.

One-half of 0.93 ft Spring Tide Range above Mean Tide Level:	(+)	0.47 ft (from NOAA Station Predictions)
Mean Tide Level above MLLW:	(+)	0.45 ft (from CO-OPS Datasheet)
MLLW above NGVD29:	(+)	0.66 ft
Mean High Water Spring above NGVD29:		1.58 ft (slightly above 1.57 ft MHHW) (the diurnal range is 0.91 ft)
(or MHWS is 0.58 ft above NAVD88)		

(4) An elevation difference of 0.13 ft exists between the permit HTL (1.71 ft) and the HTL computed from NOAA MHWS estimate (1.58 ft). If this elevation disparity were deemed significant at a project site, then NOAA CO-OPS would need to be contacted to obtain a MHWS computation from the original gage observations—in this case, from the 1978 series.

f. Transferring tidal datum elevations to a permit site. Three options would exist to transfer tidal datums from a remote gage to the project site—the permit site in this example.

(1) Perform simultaneous tide gage comparisons between NOAA gage and a temporary gage at the permit site. Given the relatively short distance (1 mile) between the gage site and the permit site, an accurate "water level transfer" of datums between the sites could be easily accomplished. In this jurisdiction, State of Florida approval for a gaging transfer would be required. Three-day staff readings would likely suffice for the datum transfer, given the short distance. For more remote project sites that are distant from the reference gage, longer comparison tide readings may be required.

(2) Run differential levels from the NOAA gage site to the permit site. In this example, a six to seven mile level line would be required around the lake to connect the sites—a 12 to 14 mile level line loop. Maintaining 0.1 ft loop closures over this distance would be problematic; thus, differential leveling over this distance may not be as effective as water transfer or GPS methods.

(3) Perform differential GPS surveys to transfer elevations from the tidal PBMs to the permit site. A static GPS baseline would effectively and accurately transfer tidal datums over this short 1-mile distance. Given the short distance, rapid/fast-static methods would also suffice. Longer baseline lengths would require full static GPS baseline observations.

On this particular site, option (3) above would likely represent the most cost-effective method for transferring tidal datums to the permit site. This transfer could be performed in less than 4 hours time. Leveling would take at minimum 2 days and water level transfer 3 days. For more distant project sites from the reference gage, option (1) would represent the preferred choice, given use of hydraulic comparisons as opposed to geodetic comparisons. Option (2) is only effective over short distances.

7-12. <u>References</u>.

33 U.S.C. 403
Rivers and Harbors Act of 1899 (Section 10)

33 U.S.C. 1344
Clean Water Act of 1972, as amended, (Section 404)

33 U.S.C. 1413
Marine Protection, Research, and Sanctuaries Act of 1972 (Section 103)

33 CFR 328
Navigation and Navigable Waters, Definition of Waters in the United States

CHAPTER 8

Monitoring Flood Protection Elevation Grades in High Subsidence Areas

8-1. Purpose. This chapter provides technical guidance for referencing project elevation grades in areas subject to relative sea level change, land subsidence, or crustal uplift. Sea level rise, coupled with subsidence, reduces protection elevations on HSPP structures; while at the same time increasing the depths of authorized/maintained navigation project grades, resulting in over-dredging. The reverse effects occur in crustal uplift regions where apparent mean sea level is falling. (Subsequent references to "subsidence" in this chapter are intended to apply to "uplift" regions as well).

a. Much of the information in this chapter is abstracted from Volume II of the "Interagency Performance Evaluation Taskforce" report (IPET 2007) that was published following Hurricane Katrina in August 2005. Volume II of this IPET study, performed jointly by USACE, NGS, and CO-OPS, focused on the development and application of a high-accuracy, time-dependent geodetic network in a high subsidence area in Southern Louisiana. This Southern Louisiana example is, therefore, applicable to other USACE project areas experiencing subsidence issues.

b. Districts involved with projects subject to significant subsidence uncertainties should closely coordinate with the NGS to ensure a suitable vertical reference framework is established to monitor elevation changes. This effort can be accomplished following NGS "height modernization" standards and specifications. In some cases, time dependent vertical networks outlined in this chapter can be established to periodically update control in unstable regions. In coastal regions, additional coordination with NOAA CO-OPS is recommended to monitor sea level datum changes in the project region. Reference marks at NOAA or local tide gages must be directly linked to NGS vertical networks in order to monitor sea level changes relative to orthometric/geodetic datums.

8-2. Background. Published elevations relative to the vertical datums in subsidence or uplift areas must be used with caution. This applies not only to NSRS or local district PBMs but also to topographic survey data derived from these bench marks (e.g., floodwall and levee protection elevations). Surveyed or published map elevations will have uncertainties due to the uneven temporal and spatial movement of the land. Thus, any geodetic or terrestrial-based elevation on a bench mark or control structure is not constant, and elevations must be periodically reobserved and adjusted for local subsidence. Likewise, hydraulic or sea level based reference datums are subject to variations due to subsidence and/or sea level change at each gage site. Sea level datums also have time varying astronomical components making their reference definition more complex than terrestrial based datums. Hydraulic low water reference datums used to define navigation grades on the Lower Mississippi River may also be subject to subsidence and other long-term variations: thus, these datums are spatially and temporally variable, and are periodically revised.

a. Long-term primary bench mark subsidence. Figure 8-1 illustrates the varying orthometric and sea level elevations recorded at a primary control bench mark at Lake Pontchartrain in New Orleans, LA. In this high subsidence area, the published orthometric elevation of this bench mark has been readjusted numerous times over the 50+ year period—with an elevation change of over 2 ft since 1951. These readjustments include datum conversions from NGVD29 to NAVD88. Evaluating settlement rates, masked with sea level rise, from legacy elevations on this bench mark is not straightforward in that leveling networks were adjusted between other unstable marks. If a tide gage had been continuously operated at this site over the 50-year period, a better estimate of relative settlement could have been made. Levee or floodwall design elevations in a high subsidence area must factor in the elevation datum uncertainties and subsidence rates at the primary bench mark.
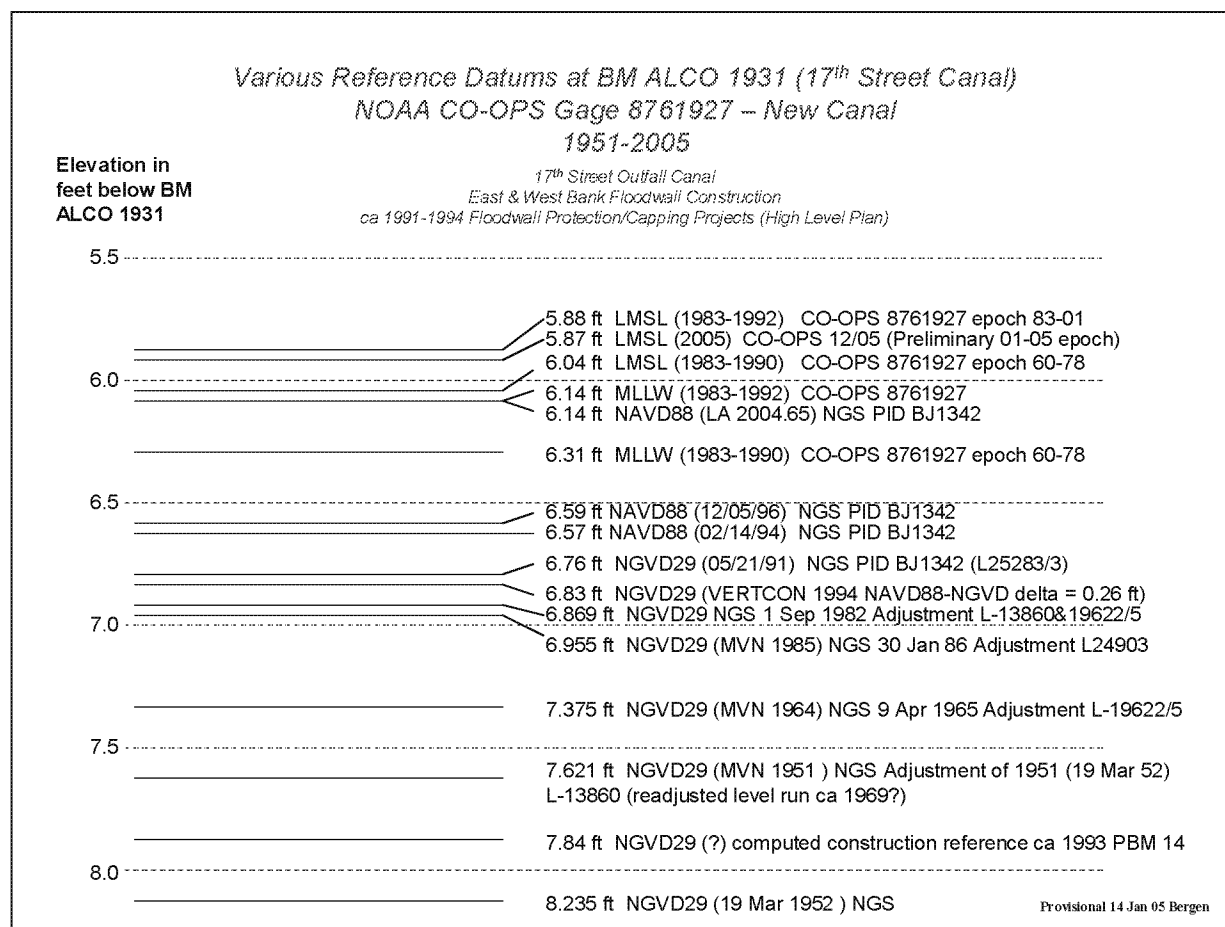

Figure 8-1. Elevations recorded at NSRS bench mark ALCO in New Orleans: 1951 to 2005.

b. Relative mean sea level. Relative or local mean sea level is the average water surface as measured by a tide gage with respect to the land upon which it is situated. Relative sea level change occurs where there is a local change in the level of the ocean relative to the land, which might be due to ocean rise and/or land level subsidence. In areas subject to rapid land level uplift, relative sea level can fall. These sea level changes result from a variety of processes, several of which can occur simultaneously. Part II of EM 1110-2-1000 (*Coastal Engineering*

Manual) lists the following processes that can contribute to long-term relative mean sea level change:

(1) Eustatic rise. Refers to a global change in the oceanic water level. Contributors to eustatic rise include melting of land-based glaciers and the expansion of near-surface ocean water due to global ocean warming.

(2) Crustal subsidence or uplift from tectonic uplifting or downwarping of the earth's crust. These changes can result from uplifting or cooling of coastal belts, sediment loading and consolidation, subsidence due to volcanic eruption loading, or glacial rebound.

(3) Seismic subsidence. Caused by sudden and irregular incidence of earthquakes.

(4) Auto-subsidence. Due to compaction or consolidation of soft underlying sediments such as mud or peat.

(5) Climatic fluctuations. May also create changes in sea level; for example, surface changes produced by El Niño due to changes in the size and location of high-pressure cells.

c. Subsidence. The subsidence effects listed above are the major contributor to elevation changes in most unstable regions. It is especially pronounced in portions of Central California (Sacramento and San Joaquin River Basins), Southwestern California, and coastal portions of Texas and Louisiana. Uplift, or apparent sea level rise, is evident in Northwest CONUS and portions of Alaska. In Southern Louisiana, subsidence is occurring at a rate of up to 0.1 foot every three years in some areas. There are many potential factors that contribute to subsidence, such as the geologic composition of the area and withdrawal of ground water and oil.

d. Subsidence at reference bench marks. Bench marks set on deep-driven rods to "apparent refusal" or bedrock will often exhibit relative subsidence to the local land surface, as shown in Figure 8-2. This relative chance is not necessarily a definitive measure of local settlement as the bench mark's refusal point may also have settled at a differing rate. Thus, the difference in elevation between the deep-driven bench mark and the TBM in the figure may or may not represent the local subsidence. The apparent subsidence of the mark on the lower left figure may be due more to local levee settlement as opposed to subsidence. The Shell Beach tidal bench mark in the figure was set in 1982 on then dry land, illustrating the rapid subsidence (or apparent sea level rise) that has occurred over the intervening 23 years in this region.
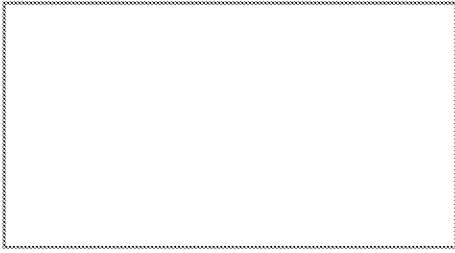
Figure 8-2. Local subsidence relative to deep-driven bench marks.

e. Monitoring elevation changes in subsidence areas with "Vertical Time Dependent Positioning" (VTDP) geodetic surveys. Changes in elevation caused by subsidence can be measured and/or periodically monitored using a combination of conventional leveling procedures and GPS techniques. Determining subsidence rates requires long-term observations and considerable analysis.

(1) Prior to the use of GPS observations, subsidence estimates were made using regional leveling networks. Presumed "most stable" bench marks were held fixed for different leveling campaigns and settlement estimates were made based on the elevation changes in these campaigns. The ability to measure accurate relative elevation differences to < 0.1 ft with GPS over long baselines (>100 miles) now provides a mechanism to connect unstable areas with reference bench marks in known stable regions

(2) As an example, Figure 8-3 shows the difference in elevations in southern Louisiana due to the differences between NGVD29 and NAVD88 adjustments along with regional subsidence. The leveling for this line was performed by NOAA in 1984 and adjusted to the NGVD29 datum at that time. In 1991, NGS adjusted the entire CONUS to the NGVD29 datum in preparation for the NAVD88 adjustment. In Southern Louisiana, an extensive "GPS Derived Height" network was completed in 2004, establishing new heights (elevations) for 85 bench marks in Southern Louisiana. The differences are shown in Figure 8-4. This "Vertical Time Dependent Positioning (VTDP)" adjustment, known as NAVD88 (2004.65), held control outside of the subsidence area to establish new NAVD88 adjusted heights for the 85 bench marks.
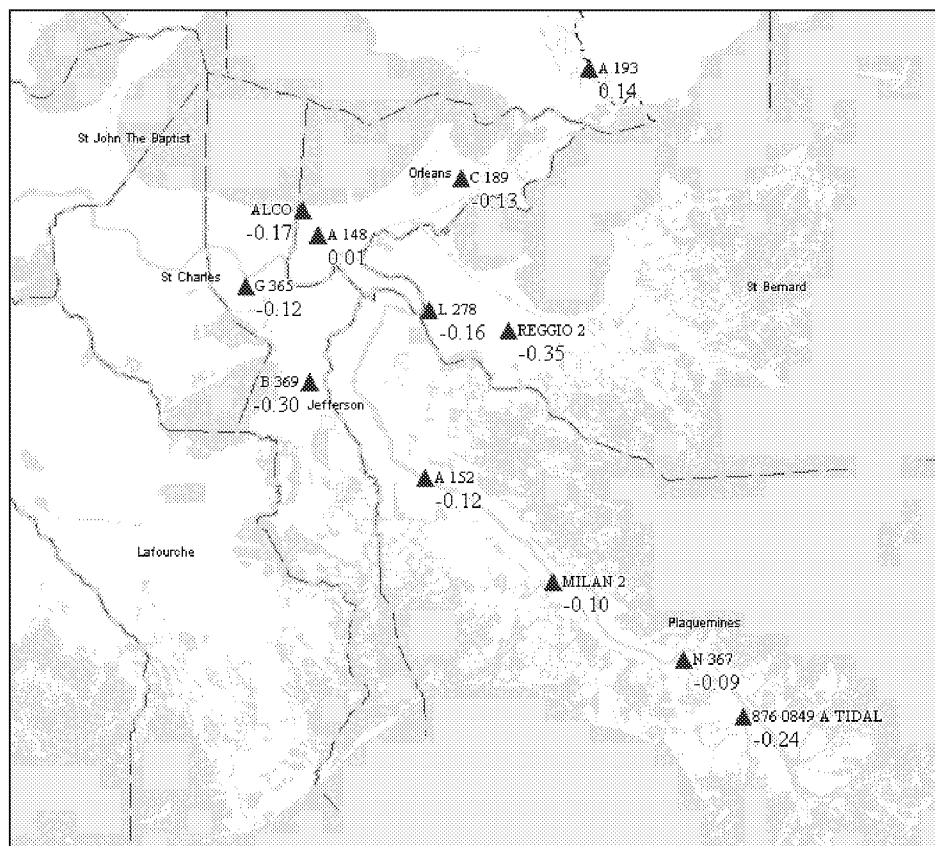
Figure 8-3. Elevation changes (ft) due to datum shift (NGVD29 to NAVD88)
and regional readjustment.

(3) Because the 1991 NAVD88 adjustment held control outside of the area, as did the NAVD88 (2004.65) adjustment, the change in the heights reflects the apparent movement of the marks between the observation periods. In order to determine the amount of subsidence from the time the original leveling was done, it is necessary to determine the amount of movement between the original adjustment and the 1991 national readjustment of the NGVD29 and then the amount of movement between the original NAVD88 adjustment and the NAVD88 (2004.65) adjustment.

f. Monitoring elevation changes in subsidence areas with water level gages. Monitoring subsidence or sea level changes on flood risk management, hurricane protection, or coastal (tidal) navigation projects requires continuous leveling or GPS surveys between water level recording gages and fixed NSRS bench marks. Geodetic surveys alone cannot determine sea level changes. Records from these gages, if reasonably well documented, can provide an independent means to investigate and determine reliable rates of local subsidence and/or validate rates determined via a VTDP geodetic survey analysis. Reference PBMs at these gages must be connected to an external geodetic network that is not impacted by subsidence. A New Orleans District study on the use of gage data in evaluating subsidence changes in Southern Louisiana is at Appendix L.
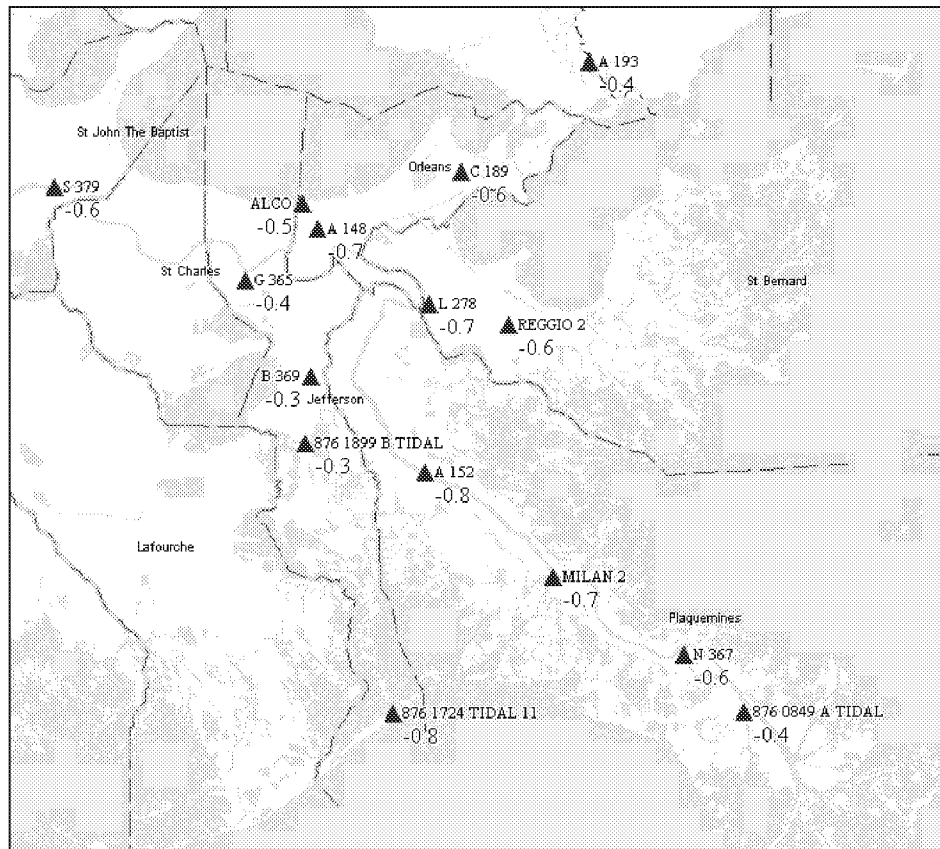
Figure 8-4. Elevation changes (ft) between NAVD88 (1996) adjustment and the VTDP NAVD88 (2004.65) regional readjustment.

g. Applications. EM 1110-2-1100 (*Coastal Engineering Manual*) notes that long-term subsidence must be factored in to the design of flood/hurricane protection structures. It states that "...the [reference] datums described above, and the reported variability of those datums, represent design criteria considerations that directly impact the expected lifetime of a project. If, for example, a coastal project is to be situated in an area of known subsidence, then design elevations need to reflect additional freeboard as a factor-of-safety consideration ..." Estimating future subsidence out 50 or more years, like sea level change, is difficult, and must be based on extrapolated trends from past geodetic surveys and water level gage data.

8-3. Development of a Vertical Time Dependent Positioning Reference Framework to Monitor Bench Mark Subsidence in Southern Louisiana. This section describes the process developed by the NGS for establishing a VTDP network in Southern Louisiana. This VTDP network is used to evaluate subsidence at bench marks in the subsidence area. A VTDP network must be continuously monitored and periodically updated—i.e., the NAVD88 (2004.65) VTDP network was subsequently updated to a NAVD88 (2006.81) epoch. The procedures outlined below are applicable to other USACE projects subject to subsidence.

a. Beginning in 2004, NGS began a series of reobservations in Louisiana for the purpose of updating the NAVD88 published heights in the region in support of hurricane evacuation

route mapping. These reobservations included both GPS campaigns and leveling observations. The GPS data were collected according to NGS standards in "*Guidelines for Establishing GPS-Derived Ellipsoid Heights: Standards: 2 cm and 5 cm*" (NOAA 1997) and in "*Guidelines for Establishing GPS Derived Orthometric Heights: Standards: 2 cm and 5 cm*" (NOAA 2005). These guidelines required a set of three 5½ hour static DGPS sessions with at least a 4 hour difference in the starting time of one session on different days. The data collected was processed using the NGS program "PAGES" and adjusted using the NGS program "ADJUST." However, prior to this adjustment, the published orthometric heights of bench marks in the Gulf Coast region from Pensacola, FL west to Houston, TX (which included bench marks occupied in the GPS reobservations in Louisiana) were updated using the most recent subsidence rates as published in NGS Technical Report 50—"*Rates of Vertical Displacement at Benchmarks in the Lower Mississippi Valley and the Northern Gulf Coast*" (NOAA 2004). These rates were applied to previous observation data and adjusted. This readjustment used 151 previously observed level lines connecting across the entire region, consisting of 16,331 bench marks. Rates of all published bench marks included in NGS Technical Report 50 (NOAA 2004) were applied. A total of 85 such bench marks were part of this reobservation campaign, as shown in Figure 8-5.

Figure 8-5. Southern Louisiana Vertical Time Dependent Network (adjustment epoch 2004.65).

b. When the GPS-derived orthometric heights were compared with leveling data at these 85 bench marks, as corrected for subsidence rates and tied to non-subsiding bench marks outside the subsidence area, there was a variety of agreements and disagreements. First, 32 of the 85 bench marks showed better than 2 cm agreement between the GPS-derived and leveling-derived orthometric heights, indicating a good estimate of subsidence rates at those points.

c. After finding the 32 points with the most reliable estimated subsidence rates, their heights were then held as stochastic constraints in a constrained adjustment of all 85 bench marks (along with fixing the heights of 4 points outside the subsidence area). The resultant adjustment of 85 heights was given the notation "NAVD88 (2004.65)", where the 2004.65 is the date in years and decimal portions of a year of the midpoint of the observation campaign. The formal accuracy estimates on these 85 bench marks fall in the 2 to 5 cm range. Note that even as these points have been adjusted to 2004.65, they are all susceptible to subsidence, and therefore it will be critical to use CORS data and possibly future re-leveling to re-adjust these heights and recompute their subsidence rates with a higher accuracy than the 2004.65 adjustment produced.

d. The NAVD88 (2004.65) adjustment, again, was not a local adjustment. It went outside of the subsidence area and held fixed what were felt to be stable bench marks. The four bench marks held fixed were: LAKE HOUSTON 2050, which is a galvanized steel pipe driven to a depth of 2050 feet; 872 9816 TIDAL 1 a TIDAL Bench mark in Pensacola, Florida; FOREST EAST BASE in Scott County, Mississippi; and M 237 in Latanier, Louisiana. A free adjustment holding LAKE HOUSTON 2050 fixed was run with the results shown in Table 8-1. The difference between the NAVD88 (1994) and NAVD88 (2004.65) reflects the apparent subsidence of the bench marks due to the procedures used in the adjustment.

Table 8-1. Louisiana VTDP Free Adjustment.

DESIGNATION	PUBLISHED (m)	ADJUSTED (m)	PUB-ADJ (m)
872 9816 TIDAL 1	1.3479	1.3741	-0.0262
FOREST EAST BASE	136.4527	136.4622	-0.0095
LAKE HOUSTON 2050	17.0714	CONSTRAINED	0.0000
M 237	20.3830	20.3422	0.0408

e. The geographical location of these fixed bench marks relative to the Southern Louisiana subsidence area is shown in Figure 8-6.

8-4. Estimating Subsidence Rates in the Southern Louisiana Region from Geodetic Observations. This section focuses on subsidence occurring at bench marks throughout the Southern Louisiana project area. A bench mark's subsidence rate may be different from that occurring in the adjacent ground—see Figure 8-2 where the deep-driven rod bench mark protrudes well above the subsided ground. Over the years, there have been several studies that have been published documenting the subsidence of New Orleans and Southern Louisiana.
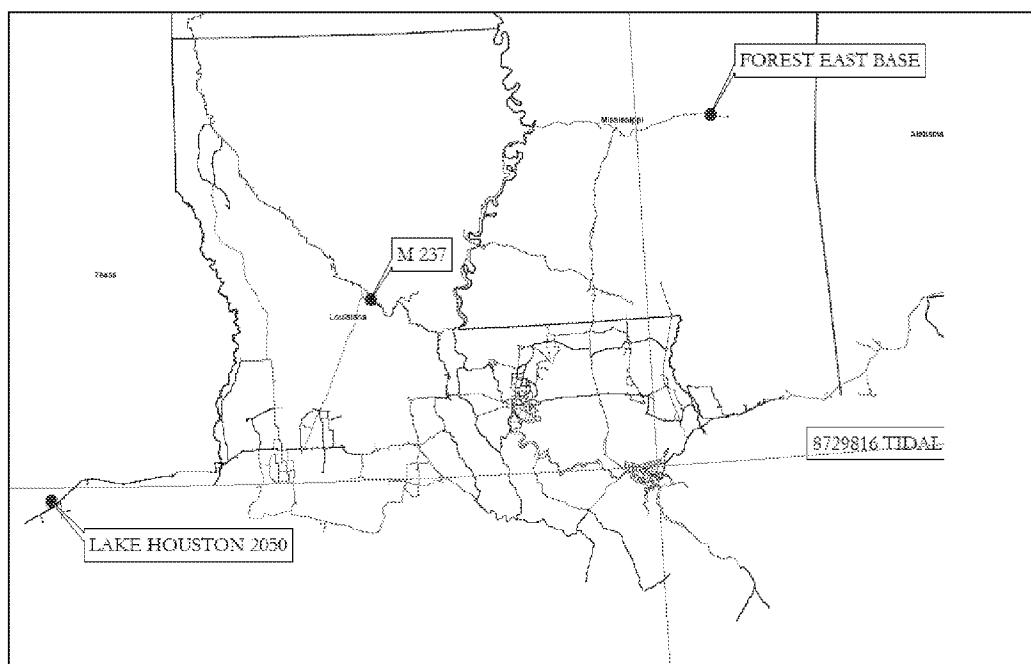
Figure 8-6. Location of fixed bench marks defining NAVD88 (2004.65) in Southern Louisiana.

a. A NOAA report "Subsidence in the Vicinity of New Orleans as Indicated by Analysis of Geodetic Leveling Data" (NOAA 1986) used three different adjustments to determine the apparent movement of bench marks in this area. This report does not show sea level rise--only the apparent movement of the benchmarks. It should also be noted that the movement reflected in this report, as well as in NOAA Technical Report 50 (NOAA 2004), reflects the movement of the mark based on leveling observations. Table 8-2 shows not only the apparent subsidence but also that the subsidence is neither linear nor at the same rate based on location and different epochs.

Table 8-2. Apparent Movement (in mm/year) without Sea Level Rise from Three Leveling Networks (1951-1955, 1964, and 1984-85) to Estimate Apparent Crustal Movement. (NOAA 1986)

PBM Designation	1985.0 – 1964.0	1985 – 1951	1964 – 1951
A 148 (AU0429)	-6.88 (21 yr)	-5.57 (34 yr)	-3.1 (13 yr)
PIKE RESET (BH1164)	-1.36	-1.59	-1.97
231 LAGS (BH1073)	-16.39	-10.90	-2.03
A 92 (BH1136)	-2.36	-2.66	-3.13

b. The rate of subsidence varies from epoch to epoch (survey to survey) due to many factors, such as compaction, removal of subsurface fluids, and geologic events. Therefore, one cannot predict future subsidence with any degree of accuracy. Table 8-3 shows the rate of

change reflected in at least two different epochs of First-Order, Class II leveling, as published in NOAA Technical Report 50 (NOAA 2004).

Table 8-3. Apparent Movement from Two Epochs of Leveling Data. (NOAA 1986)

PBM Designation	Rates of Movement (mm/year)
A 148 (AU0429)	-11.01
PIKE RESET (BH1164)	-6.99
231 LAGS (BH1073)	-16.08
A 92 (BH1136)	-7.39

c. The average rate of apparent subsidence across the region was found to be about 0.6 ft subsidence per 10 years. This indicates that elevations published in the 1960's, 70's, 80's, and early 90's may have changed even more than 1 ft. A long-term objective is to continually improve upon the vertical reference system in Southern Louisiana—e.g., NAVD88 (2004.65) was later updated to NAVD88 (2006.81). This provides a consistent framework from which the monitoring of previously constructed and proposed flood risk management and hurricane protection structures can be performed.

d. Figure 8-7 depicts estimated subsidence rates occurring at 18 benchmarks in the New Orleans region based on the adjusted elevations. The subsidence rates were computed using the difference between the published NAVD88 (2004.65) and superseded values and dividing them by the number of years between the adjustments. These rates, compared with those published in NOAA Technical Report 50 (NGS 2004), do not all agree since the adjusted elevations contain distributed errors from the adjustment computations. Therefore, Figure 8-7 illustrates the need to use unadjusted values in determining subsidence rates as documented in NOAA Technical Report 50.

8-5. Sea Level Trends in Southern Louisiana. Long term tide station records provide estimates of local relative sea level trends as opposed to the absolute rates of global sea level that are the subject of basic research in climate change. These local relative sea level trends from tide stations are a combination of global sea level variations, regional climate scale water level variations, and local vertical land movement due to local or regional subsidence. Thus the tide stations provide the information because they provide direct information on variations of water levels relative to the local land elevations.

a. Figure 8-8 depicts the apparent sea level increase (i.e., mostly subsidence) over a 60-year period at the USACE Florida Avenue gage on the Inner Harbor Navigation Canal (IHNC) in New Orleans, LA. The apparent sea level rise at this gage supports independent geodetic observations and observed elevation decreases on hurricane protection structures in this area.

PID	Designation	Rate (m/yr)	NAVD88 2004.65 (m)	Procedure	Sup/Date	Sup (m)	Sup (ft)	Leveling Year	NAVD88 (2004.65) minus Sup (ft)
BH1119	C 189	-0.016	0.63	LEVELING(2004.65)	12/5/1996	0.794	2.60	1994	-0.54
AU2163	B 369	-0.015	1.84	LEVELING(2004.65)	12/5/1996	1.975	6.48	1995	-0.44
AU2310	876 1899 B TIDAL	-0.015	0.01	LEVELING(2004.65)	12/5/1996	0.141	0.46	1995	-0.43
AU0429	A 148	-0.015	1.77	GPS OBS(2004.65)	12/5/1996	1.915	6.28	1994	-0.48
BJ1342	ALCO	-0.014	1.87	LEVELING(2004.65)	12/5/1996	2.008	6.59	1994	-0.45
AT0804	REGGIO 2	-0.012	1.52	GPS OBS(2004.65)	2/14/1994	1.714	5.62	1988	-0.64
BH1212	A 193	-0.012	0.75	LEVELING(2004.65)	2/14/1994	0.879	2.88	1993	-0.42
AU2110	G 365	-0.011	0.24	GPS OBS(2004.65)	12/5/1996	0.342	1.12	1995	-0.33
AT1390	876 0849 A TIDAL	-0.011	0.85	GPS OBS(2004.65)	8/31/2001	0.972	3.19	1993	-0.40
AT0407	A 152	-0.010	0.67	GPS OBS(2004.65)	2/14/1994	0.870	2.85	1984	-0.66
BJ3744	S 379	-0.010	4.31	GPS OBS(2004.65)	2/14/1994	4.482	14.70	1986	-0.56
AT0376	R 194	-0.008	1.39	GPS OBS(2004.65)	2/14/1994	1.554	5.10	1984	-0.54
AT0357	D 194	-0.008	1.68	LEVELING(2004.65)	2/14/1994	1.835	6.02	1984	-0.51
AT0200	MILAN 2	-0.008	-0.15	GPS OBS(2004.65)	2/14/1994	0.005	0.02	1984	-0.51
AT0332	L 278	-0.007	2.11	LEVELING(2004.65)	2/14/1994	2.253	7.39	1984	-0.47
AT0231	EMPIRE AZ MK 2 1934 1966	-0.007	-0.01	GPS OBS(2004.65)	2/14/1994	0.129	0.42	1984	-0.46
AT0247	C 279	-0.007	-0.23	GPS OBS(2004.65)	2/14/1994	-0.100	-0.33	1984	-0.43
AT0731	N 367	-0.007	0.34	GPS OBS(2004.65)	2/14/1994	0.470	1.54	1984	-0.43

Figure 8-7. Estimated subsidence rates at selected bench marks in New Orleans Region. (IPET 2007)　(Note: "Sup" = superseded)
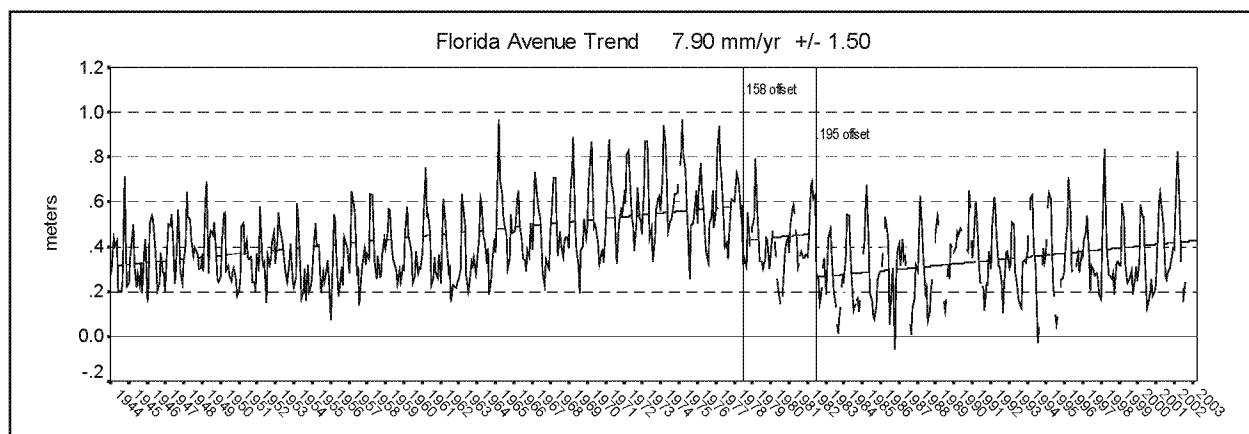

Figure 8-8. Apparent sea level rise at Corps IHNC Florida Ave. gage from 1944 to 2003. Gage zero adjustments were estimated. (IPET 2007)

b. NOAA CO-OPS has performed analyses of relative sea level trends for all of the long-term NWLON stations in their network. Unfortunately, the New Orleans area and Lake Pontchartrain are geographical areas of data gaps for locations with measurements of sea level variations necessary to estimate sea level trends with high certainty. The closest NWLON stations in this category are Dauphin Island, AL; Pensacola, Fl; and Grand Isle, LA. The analyses done for estimating relative sea level trends in the New Orleans area include using a 23-year monthly mean time series pieced together from Waveland, MS (3.52 mm/yr with a ± 2.6 mm/yr 95% confidence interval) and a 10-year monthly mean time series at New Canal, LA (3.98 mm/yr with an 95% confidence interval > ± 3.0 mm/yr). Historical once-per-day readings from long term USACE stations have also been analyzed; however, there have been many adjustments to the gages that were not readily available for this review.

(1) Analysis of the USACE record at Florida Avenue, New Orleans, LA provides a composite estimate of 7.90 mm/yr with a 95% confidence interval of ± 1.5 mm/yr.

(2) Using an assumption of similar ratio relationships of shorter period trends to longer period trends, the relative sea level trend at NWLON New Canal gage was estimated to be 6.83 mm/yr for a 23-year period (comparing with Waveland trends).

(3) By performing a difference of the simultaneous monthly mean sea levels between New Canal and Waveland, a trend fit to the differences shows that relative LMSL is rising 1.9 mm/yr faster at New Canal than at Waveland. Adding the 1.9 mm/yr rate to the 3.98 mm/yr estimate for 10-months gives an estimate of 5.88 mm/yr.

c. Although limited by the 10-year period length and with a spread of 2 mm/yr, these three independent estimates of the relative sea level trend at the New Canal gage are consistent with independent estimates of local subsidence in the region based on NOAA Report 50 (NOAA 2004), which relied on repeat geodetic surveys.

d. The results of the analyses used to estimate relative sea level trends for the Southern Louisiana study area provide corroboration of the drawbacks of estimating sea level trends from only a few decades of measurement, and the need to look at simultaneous time periods when comparing trends across a region.

8-6. <u>Seasonal Variation in Mean Sea Level in Southern Louisiana</u>. The average seasonal cycles in monthly local mean sea level can show wide variations depending on the seasonal variations in water temperature, winds, and circulation patterns currents in the nearby coastal ocean. Figure 8-10 shows four plots of monthly local mean sea levels the coastal region from Pensacola west to Grand Isle. It can be seen that there is as low progression from a single mode of a seasonal high and low sea level stand at Pensacola (high in September, low in January) to a bi-modal variation at Galveston, TX with secondary high and low in May and July respectively. Hurricane season, from June through November, coincides with the periods of high monthly local mean sea levels--this generally adds to the elevation of storm surge. Seasonal variations in the New Orleans IHNC are shown in Figure 8-9. This data were constructed by computing average water surface elevations for selected years at the USACE Florida Avenue gage. Elevations are in feet and are referred to approximate LMSL or NGVD29 (1983 adjustment). Figure 8-10 clearly shows a

quarter-foot bias in average surface elevation during the fall hurricane season. Hydrodynamic modeling, risk analysis, and design criteria need to consider this seasonal bias in evaluating flood protection elevations.
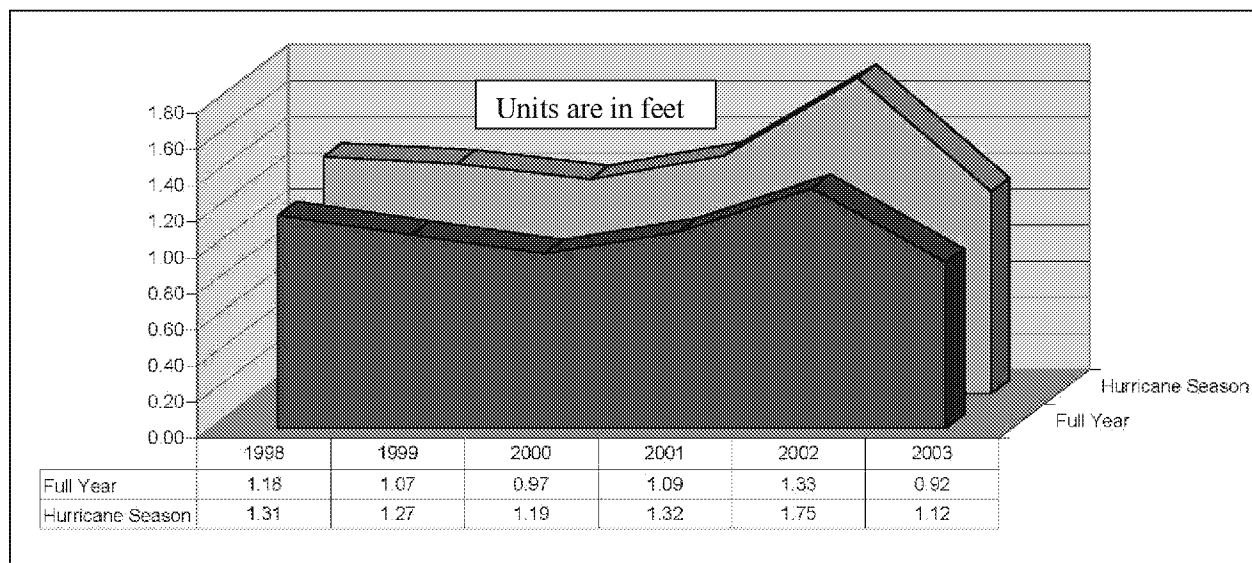
	1998	1999	2000	2001	2002	2003
Full Year	1.18	1.07	0.97	1.09	1.33	0.92
Hurricane Season	1.31	1.27	1.19	1.32	1.75	1.12

Figure 8-9. Seasonal variations (in feet) at the New Orleans IHNC Florida Avenue gage.
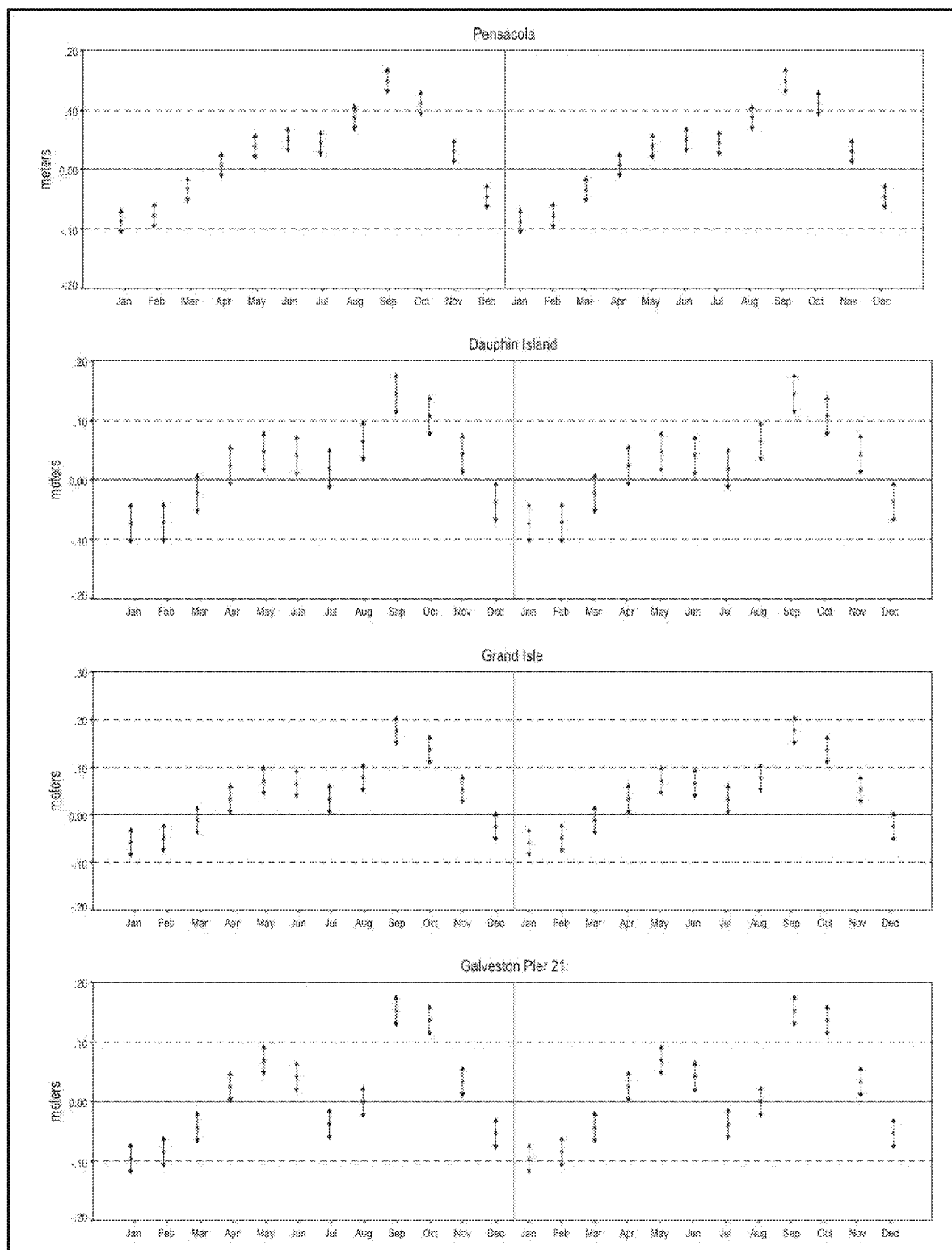
Figure 8-10. Monthly local Mean Sea Levels from Pensacola to Galveston.
(in meters above LMSL)

CHAPTER 9

Checklist for Assessing Project Datums and Elevation Uncertainties through Project Phases

9-1. Purpose. This chapter provides guidance for evaluating the adequacy of elevation grades and reference datums through the life cycle of a project. This includes evaluations or assessments to ensure the project is referenced to the current NSRS and/or NWLON framework. Procedures for estimating project grade or depth measurement uncertainties are also outlined.

9-2. Planning and PED Phases—Reference Datum Checklist. During the planning and/or detailed design phases (e.g., PED), water level datums, geodetic datums, and topographic elevation references shall be clearly defined and established throughout the project area. This entails setting Primary Project Control Points (PPCPs) at a spacing sufficient to densify supplemental (local) control for subsequent engineering and construction surveys, as outlined in previous chapters of this manual. All PPCPs must be published in the NSRS. The project area includes not only the planned location of a flood protection structure but also related flood plain mapping on the protected side of the control structure and perhaps hydrographic surveys on the flood side. Navigation projects may include external confined disposal and beach renourishment sites. These design reference surfaces must be established prior to performing basic site plan mapping, aerial mapping, LIDAR elevation mapping, hydrographic surveys, geotechnical investigations, and related preliminary design requirements. The main issues to be evaluated and resolved during the preliminary planning and/or design phases are listed in Table 9-1.

Table 9-1. Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Establish Primary Project Control PBMs	Use existing (published) NSRS PBM or survey new PBM and submit/publish in NSRS—see Chapter 3
Reference datums	NAD83, NAVD88, & hydraulic/tidal
Accuracy required	see nominal standards in Table 3-1
Density of Primary Control PBMs	see Chapter 6 (Inland projects)
Recommended survey procedures	see Chapter 3
PPCP satellite visibility	Verify horizon clearances

Table 9-1 (Continued). Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Local construction reference PBMs (LPCP)	Survey connections made directly from PPCP PBMs
Datum	NAD83, NAVD88, & hydraulic/tidal
Local construction datum	Note relationship to NSRS (NAVD88)
Density of LPCPs	Ensure spacing within leveling or RTK ranges to project
Local relative accuracy	see Chapter 3
PBMs indicated in contract documents	minimum of three required for construction plans & specs
Legacy Datums	Document reference to NSRS (NAVD88)
Protection Grade Elevation References	Ensure referenced to NSRS (NAVD88) from PPCP/LPCP ties
Subsurface investigation boring reference elevation	NSRS/NAVD88—connected from PPCPs or local PBMs
Site plan mapping reference datums	NAD83 and NAVD88 (current adjustments and epochs) and Local Station-Offset system
Detailed topographic site plan accuracies (hard features, ground shots, etc)	See Chapter 3 (total station or RTK methods relative to PBMs)
Hydraulic/tidal gage reference PBMs	Directly referenced to river/tidal gage reference datum
Minimum number of gage reference PBMs	3 (one PBM must be connected to/published in NSRS)

Table 9-1 (Concluded). Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Navigation MLLW datum modeling	VDatum or spatially interpolated—document model
Navigation RTK base station	Documented in plans and published in NSRS
Navigation tidal PBM calibration points	Documented in plans and connected to CO-OPS network
Metadata	Design memorandums, project drawings, CADD files, studies, reports, flood profile diagrams and related framework documents contain full and complete metadata on the reference elevation datum, primary project control PBMs, and local construction control PBMs; including the relationships and estimated accuracies of legacy reference datums, bench marks, and designed protective elevations.

9-3. Construction Phase Checklist.

a. Minimum construction stakeout criteria. Local horizontal alignment and vertical control PBMs established during the detailed design phase and shown on the contract plans shall be thoroughly verified during the initial construction stakeout. This verification entails checks to a minimum of three PBMs shown on the contract drawings. Checks between the local reference points should generally agree to within ±0.05 ft. Checks on horizontal alignment control points or bench marks exceeding these tolerances shall be thoroughly investigated and resolved prior to construction stake out. The government construction inspector shall review in progress (on site) initial construction stakeout work and shall thoroughly review the contractor's stake out notes for both the basic control check and the site stake out.

b. Machine control system calibration. Machine control positioning systems on graders and bulldozers must be verified on site to ensure horizontal and vertical grading references check with fixed project control bench marks. Machine control RTK networks must also be adequately "site calibrated" prior to excavation or grading, ensuring fixed calibration bench marks surround the construction site.

c. Verification of as-built floodwall cap elevations. Post-construction profile or topographic surveys of floodwalls shall be made to verify as-built controlling elevations and horizontal location. These surveys may be performed using total stations, levels, or RTK methods. Surveys must originate from the reference control PBMs shown in the contract plans. Elevations of sheet pile or floodwall caps should be recorded to the nearest 0.1 ft.

d. Navigation project control verification. RTK or RTN horizontal positioning calibration shall be checked or site calibrated at independent PBMs. RTK/RTN water surface elevation measurements shall be calibrated to the local river or tide gages. In tidal areas verify the latest MLLW gradient model is being used. Levels should be run between tidal reference PBMs to verify stability. Staff gages should be set by leveling to a minimum of two reference PBMs.

9-4. Post-Construction (Operation and Maintenance) Phase—Periodic Reassessments or Evaluations of Controlling Reference Elevations. Periodic reevaluations of project reference elevations and related datums shall be included as an integral component in the various civil works inspection programs of completed projects. The frequency that these periodic reevaluations will be needed is a function of estimated magnitude of geophysical changes that could impact designed protection grades. Most USACE projects are in relatively stable areas and can be evaluated at less frequent intervals. Some criteria for determining resurvey frequencies might include: (1) protected population areas, (2) known insufficient datums, (3) known settlement problems, (4) known subsidence or crustal uplift, (5) District or sponsor priority, (6) type for flood protection structure, or (7) structure height. Navigation project grades or flood protection elevations that are referenced to tidal datums will have to be periodically coordinated with and/or reviewed by NOAA CO-OPS to ensure the latest tidal hydraulic effects are incorporated and that the project is reliably connected with the NSRS. For dams, levees, and related structures, a complete reevaluation of the vertical datums should be conducted at the frequency specified in the O&M Manual for the project; typically ranging from 2 to 5 years in high subsidence areas to 10 or more years in stable areas. Any uncertainties in protection levels that are identified during the inspection should be incorporated into any applicable risk/reliability models developed for the project. Technical guidance on periodic inspection monitoring surveys is found in EM 1110-2-1009 (*Structural Deformation Surveying*).

a. Reference bench mark verification. Periodic resurveys shall be performed relative to the PPCPs established for the project. The NSRS datasheet shall be reviewed to determine if NGS has revised the elevation for the primary mark. The stability of the PPCPs shall be verified by GPS observations or differential level runs to adjacent NSRS reference bench marks. Checks to ± 0.1 ft would be a reasonable tolerance. The PPCP should normally be used as the base station when GPS RTK surveys are performed at the project site.

b. Topographic survey methods. Topographic surveys of floodwall caps, levee or floodwall profiles, inverts, pump stations, etc. should generally meet the tolerances indicated in Chapter 3, which are relative to the LPCPs and/or indirectly to the NSRS PPCPs. Differential leveling (spirit or digital), GPS RTK, or total station methods should yield ± 0.1 ft relative accuracies on surveyed points relative to LPCPs. Reference also topographic surveying methods in EM 1110-1-1005 (*Control and Topographic Surveying*).

c. Profile surveys of levee grades and floodwall caps. Periodic topographic surveys of levee and floodwall elevations shall be performed to verify the current protection elevations. Either differential leveling or GPS RTK methods may be used—RTK normally being the most efficient method given 3D coordinates are directly observed at each point. Shot points are taken at 50-ft or 100-ft intervals along the structure, breaks in grade, gate structures, monoliths, and at other features as designated.

d. Topographic sections on protected or flood sides of floodwalls. Floodwalls set atop or around bridges, levees, pump stations, and other facilities may require periodic topographic surveys of the surrounding berms, revetments, chords, or water depths. Subsurface hydrographic surveys may be required in adjacent canals or rivers to check for scour into the levee revetment or floodwall base. The density of such surveys will depend on the potential scour or settlement being monitored. Typically, 50- to 100-ft sections will be surveyed using standard topographic survey methods, such as GPS RTK.

(1) Hydrographic surveys of deeper water on the flood side can be performed following the techniques outlined in EM 1110-2-1003 (*Hydrographic Surveying*). In shallow river or canal areas (i.e., < 15 ft water depth), standard leveling or total station topographic survey methods may be used with a 25-ft expandable level rod. Typical cross-section spacing is 50 ft or 100 ft c/c.

(2) Acoustic depths may be taken from a boat using inexpensive single-beam survey methods. If 100% bottom coverage is required to evaluate scour or other anomalies in a floodwall or levee footing, then either multi-transducer or multibeam survey systems may be employed, depending on water depth and other factors. Other high-definition acoustic devices may also be used.

e. Deformation and deflection measurements. Many of the precise survey procedures used for large dams outlined in EM 1110-2-1009 (*Structural Deformation Surveying*) may be applied to levees and floodwalls—on an isolated basis given the large geographical extent of floodwalls as compared to dams. This would include precise differential leveling to monitor regional subsidence and settlement, and crack or monolith lateral movement using micrometers. A number of options exist to monitor relative (internal) horizontal deflections of individual floodwall sections. Overall (global) lateral deformation or translation requires monitoring from undisturbed permanent reference points.

f. Navigation and coastal shore protection structures. Coastal navigation, shore protection, and hurricane protection projects need to be periodically evaluated to check for updates to the reference sea level datum. This is normally performed during the development of maintenance dredging plans & specifications.

(1) A periodic assessment of these projects is intended to verify (1) that the design/constructed sea level reference datum is current (i.e., latest tidal datum epoch and model) and (2) that the local project control has been connected with the latest NSRS (NAVD88) adjustment.

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(2) Many shore protection projects were originally designed to sea level datums based on interpolated or extrapolated references from gages. Depending on the type of gage, tidal range, and the distance from the gage, this interpolation or extrapolation may no longer be valid or sufficiently accurate—i.e., generally within ± 0.25 ft of the reference water level datum. With sea level rise, the crest elevation of structures may be below that originally designed. However, the original design documents should be checked to verify that allowance for sea level rise was considered in the design elevation and is consistent with the current condition.

g. Coastal navigation project reference datums. Reference tide gages should be checked for periodic datum updates or corrections by NOAA CO-OPS. Updates to VDatum models should also be checked to make sure the latest revisions are accounted for in the model.

h. Checklist. Table 9-2 summarizes some of the items that should be evaluated during periodic inspections or resurveys of levees and floodwalls.

Table 9-2. Summary of Requirements for Referencing Levee and Floodwall Elevations during Post-Construction Maintenance (Periodic Evaluation) Phase.

Post-construction Operation & Maintenance	Periodic inspection and verification of reference hydraulic/tidal and geodetic NSRS datums, subsidence, and sea level changes
Verification of Primary/local PBM relative to NSRS regional network	Check tolerance: ± 0.1 ft (3D)—see criteria in Chapter 3
Topographic inspection survey density:	
Floodwall cap profile surveys	25 to 100 ft shot points (typical) plus breaks in grade
Cross-section topo/hydro surveys	50 or 100 ft c/c typical
Resolution	± 0.1 ft (3D) typical

9-5. Sample PED Evaluation Report on a Hurricane Protection Project's Reference Datums. The following example report contains excerpts from an evaluation of reference datum connections in a New Orleans District Design Report. This evaluation checklist and report reviewed the reference datums used for various engineering disciplines covered in the report. The initial checklist indicates areas that will require additional field survey or design review effort.

Comprehensive Evaluation of Project Datum--Quality Control Checklist (New Orleans District)

Title: LPV-12.2, Hurricane Protection Project, Jefferson Lakefront, Fronting Protection, Duncan Pumping Station, Jefferson Parish, Louisiana - Design Report

Prepared by:	*mh*
Checked By:	*rmf*
Date:	*31-May-07*

General Checklist

[No]   *Gages Referenced To both NAVD88 and Latest Epoch Tidal Datum (MLLW, LMSL)*
[???]   *Gage Inspection Current*
[No]   *Do Plans Document 3 PBMs*
[No]   *Is A PBM Tied To NAVD88 and Tidal Datum*
[N/A]   *Is Navigation Project Tied To MLLW*
[???]   *Has Subsidence and Sea Level Rise Been Considered*
[Yes]   *Are Units Specified (US Survey Foot)*
[Yes]   *State Plane Zone Specified*
[No]   *Do Project PBMs Indicate Epoch, Datum, Description, Elevation*

Comments [Excerpted]:

Executive Summary

3.0 Site Survey Plan
This is very good. It makes it clear which horizontal and vertical systems were used in the preparation of the topographic survey and also makes it clear that older pump station plans are referenced to Cairo 1910 and incorporated into the report for "informational purposes only."

Design Report

3.0 Site Survey Plan
Good. Includes additional information that confirms Geoid03 (revised for South Louisiana in Oct '05) was used in GPS processing. Refer to Appendix C below for more detail.

10.0 Preliminary Cost Estimate
Drawings in 10.2 Demolition indicate "NGVD" in margin. Should be NAVD88 (2004.65)

Appendix A - Scope of Work

7. Site Surveys and Mapping
All federally funded Hurricane Protection, Flood Control, Shore Protection, and Navigation projects require documentation of the following:
1) Reference to accurate and current hydraulic/tidal datum (e.g., LWRP, LMSL, MLLW) based upon an adequate gage network with ties to NOAA using latest tidal epoch.

2) Three bench marks at each gage relative to NAVD88 (latest adjustment/epoch with at least one PBM directly tied to the NSRS) and from which rigorous gage inspections are performed and documented (bench mark ties to 3rd order)

3) Reference/relationship from latest epoch of NAVD88 to construction/design datum if other than NAVD88 or current hydraulic/tidal datum (e.g., relationship from NAVD88 to MLG, NGVD29, Cairo, etc.).

A total of three bench marks needs to be identified or established at the project site in accordance with CEMVN-ED-SS-06-01, "USACE New Orleans District Guide for Minimum Survey Standards for Performing Hydrographic, Topographic, and Geodetic Surveys" and the location, identification and elevation of these bench marks needs to be shown on all relevant project sheets/drawings (see Appendix C note below).

8. Geotechnical Explorations, Test, and Analysis
Soil Borings and Cone Penetrometer test to be tied to baseline with station and offset and X/Y and elevation given with respect to project reference systems (see Appendix D note below).

Appendix B - Plates

Plate #4 - Confirm survey baseline referenced to Geodetic North Azimuth and add stationing, PI Coordinates, source, ID, etc. or remove completely.

Plate #27
"* Existing elevations are per the design drawings.
Actual as-built elevations are +/-0.8' lower"
This is a very good and helpful note.

Appendix C - Site Survey Plan

Need to show site map and data sheets for control points "082806GD" and NGS "BUICK" (pictures, description, location, references, elevation, etc.) and establish a third bench mark with all of this information.

Appendix D - Geotechnical Investigation

Only two soil boring logs shown (of eight proposed). Ground elevations for two soil boring locations shown on log, margin info indicates NGVD (confusing/ambiguous).

*Station/offset info for proposed soil boring and CPT test sites unclear. Need coordinates
and elevations of all soil boring and CPT sites.*

*General - Include statement: All surveys shall be conducted in accordance with CEMVN-
ED-SS-06-01, "USACE New Orleans District Guide for Minimum Survey Standards for
Performing Hydrographic, Topographic, and Geodetic Surveys" and shall be submitted to
ED-SS. The guidance is available at
http://www.mvn.usace.army.mil/ed/edss/surveyingguidelines.asp*

9-6. <u>Elevation Uncertainty Estimates of Reference Grades</u>. The surveyed elevation of a flood
protection structure or navigation grade has an uncertainty due to the propagated errors of all the
uncertainties in the components that derived the elevation. These include the regional geodetic
PPCP datum uncertainties, hydraulic or tidal datum uncertainties, water level gage references,
local LPCP datum uncertainties, topographic/hydrographic survey errors, feature irregularities,
etc.

a. For example, if the PPCP for a levee project has an estimated NSRS accuracy of ± 0.2
ft, and topographic surveys or the levee profile are performed through local LPCPs on the levee,
then the resultant NSRS accuracy of a ground shot atop the levee (or a first-floor elevation in the
flood plain) could propagate to as much as ± 0.5 ft. Likewise, the resultant elevation accuracy of
a navigation project grade or HSPP structure elevation depends on reliability of the tidal datum,
sea level change estimates, and the depth measurement process.

b. Uncertainties in navigation depths will normally range between ± 0.5 ft and ± 1 ft, or
larger in some projects. These propagated uncertainties must be estimated for each project and
factored in to the risk analysis or design of a protection grade or navigation channel design
grade—see EM 1110-2-1619 (*Risk-Based Analysis for Flood Damage Reduction Studies*). Table
9-3 lists typical elevation uncertainty estimates for inland and coastal projects.

Table 9-3. Typical Elevation Uncertainty Estimates of Gages and Project Features Relative
to NSRS.

Feature	Elevation Uncertainty Project (Standard Deviation 95%)
River Gages	
Gages directly connected to NSRS based on direct leveling or DGPS satellite observations	± 0.05 ft to ± 0.2 ft
Gages on legacy datums with firm (published NSRS) relationships	± 0.15 ft to ± 0.4 ft
Gages on legacy datums without firm (or unknown) connections to national vertical network	± 0.5 ft to ± 2.0 ft

Table 9-3 (Concluded). Typical Elevation Uncertainty Estimates of Gages and Project Features Relative to NSRS.

Feature	Elevation Uncertainty Project (Standard Deviation 95%)
Topographic Feature Elevations (Propagated Errors)	
Levee/floodplain/first-floor elevations based on direct connections with current NSRS bench marks	± 0.2 ft to ± 0.3 ft
Levee/floodplain/first-floor elevations based on legacy datums and uncertain PBM origins	± 0.5 ft to ± 1 ft
Levee/floodplain/first-floor elevations based on legacy datums but firmly related to current NSRS vertical network	± 0.3 ft
Coastal Project Grades	
Tide Gages (function of period of record, epoch, etc.—see Chapter 4)	± 0.2 ft to ± 0.5 ft
Tidal model at project site hydrodynamically modeled to local NOAA LMSL datum	± 0.1 ft
Tidal model at project site estimated based on unknown or outdated tidal datum (uncertainty function of tide range and distance from original gage)	± 0.2 ft to ± 0.5 ft
Navigation channel depth or HSPP grade (propagated error)	± 0.5 ft to ± 1.0 ft

c. Appendix M *(Uncertainty Model for Orthometric, Tidal, and Hydraulic Datums for use in Risk Assessment Models)* discusses methods for estimating overall datum and survey uncertainties on USACE project grades, and the statistical factors that should be considered in arriving at risk assessments associated with datum uncertainties. This appendix contains practical examples of the factors (such as those in Table 9-3) that must be incorporated in datum uncertainty computations.

9-7. Computing Elevation Uncertainties in the Design of Flood Protection and HSPP Structures. Uncertainty is defined as the result of imperfect knowledge concerning the present or future state of a system, event, situation, or (sub) population under consideration. Datum and resultant elevation uncertainties of reference PBMs and gages must be factored in the design of protection elevations on inland or coastal flood protection structures. Uncertainty "allowances" also factor in to risk-based design of protection elevations, which involves estimating the probability and

severity of undesirable consequences of a failure, e.g., loss of life, threat to public safety, environmental and economic damages. Risks associated with datum and subsidence uncertainties would involve potential overtopping during flood stages. General guidance on these design considerations is summarized below.

Loss of protection due to lowering of the top of flood barrier relative to design water levels shall be accounted for in any flood risk management project with site geology that is undergoing long term regional settlement [subsidence] and/or on coastlines where future sea level rise is occurring. For the system to be reliable, the top of the flood protection must be able to provide the required design height over the service life of the project. In areas where subsidence is a concern, a comparative analysis shall be performed ...To ensure reliability of the system, and to account for local settlement caused by the weight of levees, or from general lowering of an area relative the water level due to regional subsidence and/or sea level rise, flood risk management projects shall be initially constructed to a height sufficient to maintain the required height for all future conditions. Flood risk management projects shall also be constructed to the design level for current conditions with allowance for raising in the future to meet design heights as settlement and/or subsidence occurs.

a. An additional freeboard allowance can be estimated that will account for geodetic datum uncertainties and long-term subsidence. The floodwall depicted in Figure 9-1 depicts freeboard allowances for uncertainties in the regional geodetic datum and regional subsidence. These allowances may be estimated from the ranges shown in Table 9-3 and from the uncertainty estimates listed in Table 9-4 in the following section.

b. Application of these uncertainty allowances are outlined in EM 1110-2-1619 *(Risk-Based Analysis for Flood Damage Reduction Studies).* .
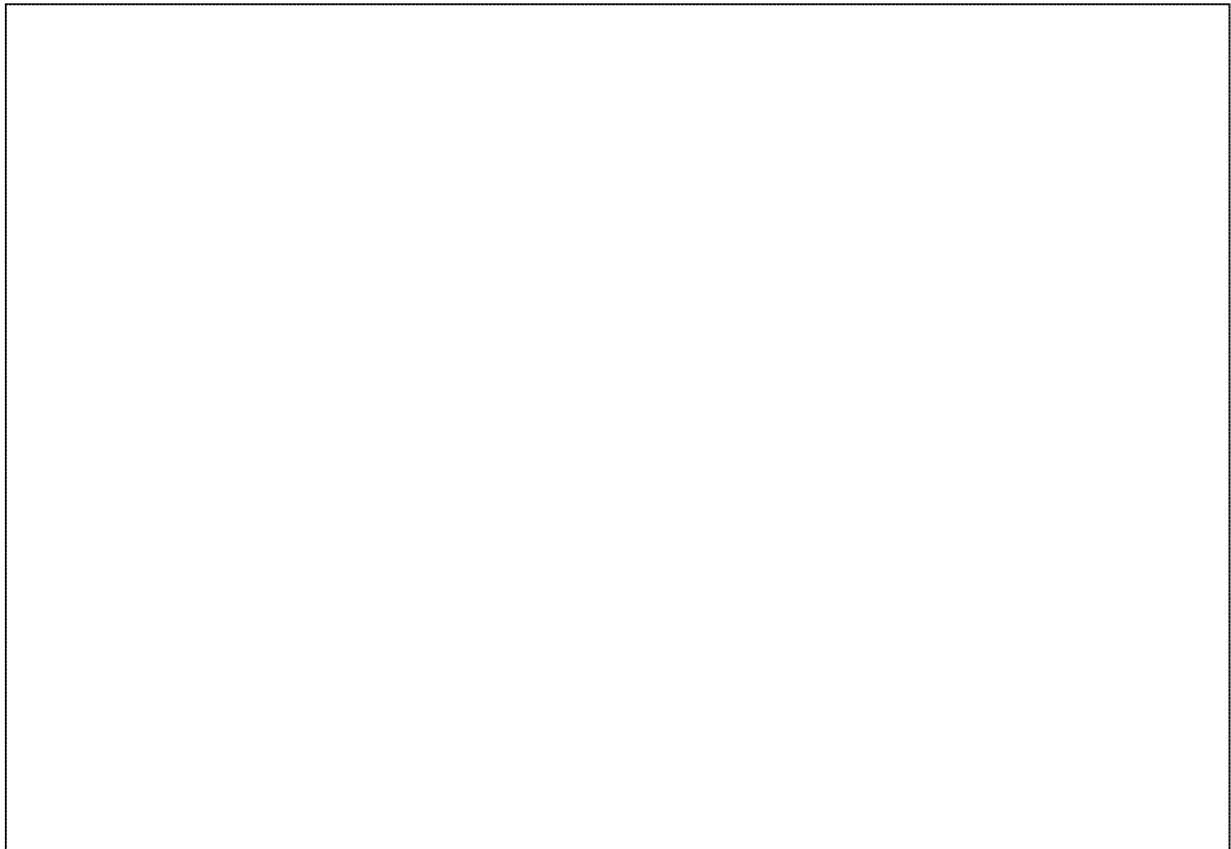
Figure 9-1. Allowances for geodetic datum and subsidence in risk-based design.

9-8. Site Information Classifications and Requirements. Table 9-4 provides general site information classifications for reference datums, based on various levels of potential adverse site conditions. These classifications apply to the design of new protection structures or an evaluation of existing projects. "Well-Defined" or "Ordinary" classifications are considered acceptable. "Limited" site information will require additional field survey data. Datum or subsidence uncertainty estimates shown in the table should be factored into design risk assessment models and floodwall height overbuild computations.

Table 9-4. Site Information Classifications and Uncertainty Estimates (95% Confidence Levels)
of the Primary Project Control Point (PPCP).

Condition	Well-Defined	Ordinary	Limited
Connection to existing NSRS PBM	1st Order PBM in NSRS	2nd Order NSRS PBM	3rd or 4th NSRS PBM
Surveyed connection method with NSRS	1st/2nd Order differential levels	2/5 cm NGS GPS standards 3rd Order levels GPS CORS/OPUS	GPS RTK or unknown method
Reference orthometric datum	NAVD88	NAVD88	NGVD29
Published in NSRS	Yes	Yes	No
Estimated network orthometric datum accuracy relative to NSRS	± 0.02 ft to $< \pm 0.10$ ft	$> \pm 0.10$ ft to $< \pm 0.25$ ft	$> \pm 0.25$ ft
Estimated regional hydraulic/tidal water level datum accuracy at gage reference PBM	± 0.05 ft to $< \pm 0.10$ ft	$> \pm 0.10$ ft to $< \pm 0.25$ ft	$> \pm 0.25$ ft
Uncertainty in 50-year subsidence forecast predictions (95%) in high subsidence areas	$< \pm 0.1$ ft	$> \pm 0.1$ ft to $< \pm 0.5$ ft	$> \pm 0.5$ ft
Uncertainty in 50-year sea level forecast predictions (95%)	$< \pm 0.1$ ft	$> \pm 0.1$ ft to $< \pm 0.5$ ft	$> \pm 0.5$ ft

9-9. Estimating Uncertainties on Coastal Navigation Project Grades. The design navigation
grade or required dredging template needs to contain an allowance for uncertainties in the
reference datum, tidal models, and survey accuracies. This allowance is dependent on a
statistical analysis of the "total propagated uncertainty" (TPU) of individual depth measurements
made by the acoustic measurement system, along with estimated hydrodynamic, meteorological,
and environmental conditions occurring at a specific project site. Statistical uncertainties in the
overall depth measurement process at a specific project site should be reviewed and evaluated
during the PED phase. These will include local system variables (e.g., positional uncertainties,
acoustic calibration precisions, vessel motion correction, acoustic depth resolution, sound

velocity and outer beam refraction, etc.) and other systematic biases (tidal phase variations, tidal MLLW modeling variations, etc.) that may be present in the propagated depth error budget—TPU.

a. Indeterminate biases. Indeterminate biases include biases in tidal models, tidal epoch latencies, reference datum biases, tidal bench mark settlement, sea level change, acoustic bottom reflectivity, reference datum adjustments, geoid readjustments, and other largely indeterminate factors. These are biases that are difficult or nearly impossible to measure or correct for. They are generally not factored in dredge clearance assessment. This is because these biases are present in all repeated surveys over the project, assuming the same vertical reference tidal bench mark is used on a given project. They do, however, enter into the estimated uncertainty of a reported channel clearance to the public and cost estimates for dredging.

(1) For example, sea level rise occurring between tidal epoch updates could be as much as 0.2 ft. Thus, the MLLW datum at the reference bench mark would have a constant bias of 0.2 ft and the reported channel clearance constantly off by that same amount. This equates to overdredging the project by a constant 0.2 ft, which may have significant budget impacts.

(2) The use of outdated or undefined local reference datums will also cause systematic biases in the maintained or reported project depth. Datum biases of upwards of 2 ft have been known to occur, resulting in incorrectly reported or interpreted channel clearance depths.

(3) Tidal bench mark elevations used to reference measurement, payment, and clearance surveys at a project are also subject to uncertainties. The stability of the bench mark could be subject to regional settlement or uplift. The MLLW datum has an uncertainty dependent on the length of the time the gage was in place, the distance from a primary gage, and other factors. The uncertainty of the computed MLLW datum at a gage site can range from ± 0.1 ft to as much as ± 0.25 ft—see Chapter 4. It is also assumed that a primary reference bench mark is used to control all surveys performed at a given project site. If different bench marks are used, and inconsistencies between these bench marks exist (height or MLLW datum), then these errors would be propagated into the TPU estimates. An example would be uncertainties in a tidal zoning model.

(4) Tidal datum variations over a project may be subject to uncertainties if not minimized by some form of hydrodynamic modeling, such as those used in developing VDatum tidal datum fields.

(5) Geoid undulations occurring over a project must be modeled if RTK methods are used to measure the water surface elevation. Geoid model uncertainties in coastal areas are typically at the 1 to 3 cm range, with predicted uncertainties slightly larger (5 cm) in offshore entrance channels. There are no practical methods of refining the model in offshore models; however, since these errors are systematic to all users of the same model, survey repeatability (or more importantly, reproducibility) is not impacted.

(6) The accumulation of these global uncertainties can range from 0.1 to 0.5 ft. The addition of these global uncertainties can propagate to an overall uncertainty in the reported

project clearance. For example, a project with an estimated local survey confidence of ± 0.25 ft relative to a fixed bench mark/gage and an estimated global uncertainty of ± 0.25 ft would have an overall uncertainty of nearly ±0.4 ft. Given these uncertainties, reporting project clearances to an implied 0.1 ft confidence level is problematic.

b. Water surface correction uncertainty due to unmodeled tidal phase lags. Aside from vessel motion corrections (roll, pitch, yaw, heave), the largest portion of the depth error budget (TPU) is attributable to unmodeled tidal phase lags—i.e., surface slope gradients between the reference gage and the project site. This error is significant is tidal estuaries, rivers, or when inshore gage readings are extrapolated out into a coastal entrance channel—see Chapter 4. If RTK-derived water surface elevations are measured, coupled with GPS-aided IMU systems to correct vessel motions (e.g., POS/MV), then the uncertainty of the water surface elevation measurement at the project site may be estimated.

c. General measurement uncertainties. Uncertainty estimates in the design and maintenance of navigation grades in a typical navigation project of limited geographical extent are summarized in Table 9-5. This table differentiates between the survey procedures used to measure the water surface at the offshore project site—(1) unmodeled surface elevation extrapolation from a shore-based tide gage or (2) direct RTK surface elevation measurement at the project site. This table is not inclusive of all the measurement factors that make up a depth measurement—see the TPU factors in Figure 9-3.

Table 9-5. Estimated Uncertainties in Measuring Navigation Project Grades in a Typical Navigation Project.

Measurement Factor	Uncertainty Range
Tidal gage MLLW datum accuracy	0.1 - 0.2 ft
Tidal epoch latency (update lag during 19-year period)	0.05 - 0.1 ft
Projected gage/tidal PBM elevation (RTK):	
RTK geoid prediction	0.1 - 0.2 ft
RTK accuracy	0.1 - 0.15 ft
Projected gage/tidal PBM elevation: (extrapolated from gage to work site)	
MLLW range gradient (unmodeled/estimated)	0.1 - 0.3 ft
Tidal phase lag (gage to work site)	0.2 - 2 ft +
Acoustic depth measurement uncertainties:	
Depths < 15 ft	0.05 – 0.1 ft
Depths 15 ft to 40 ft	0.1 – 0.3 ft
Depths > 40 ft	0.3 – 0.5 ft

(1) The applicable uncertainties in Table 9-5 are statistically propagated to determine the resultant uncertainty of a depth measurement and uncertainty in the dredged clearance estimate.

(2) As an example, given a Gulf Coast 45-ft deep-draft navigation project located 5 miles distant from the reference tide gage. The reference gage datum computation was based on 90 days of observations 30 years ago. The tide readings at the gage are extrapolated out to the project site without any tide range or phase correction. The mean tide range is 8 ft at the offshore project site and 6 ft at the gage. The phase lag between the project site and gage is estimated at 45 minutes. The TPU of the measured grade would be estimated as follows:

Estimated Uncertainty Factor in TPU	Uncertainty in ± ft (95%)	
Tidal gage MLLW datum accuracy	0.3 ft	(Chapter 4)
Tidal epoch latency (update lag)	0.05 ft	(1993 to 2009)
Extrapolated (projected) surface from gage		
MLLW range gradient	0.2 ft	(unmodeled MLLW reference)
Tidal phase lag (average ebb/flood)	0.7 ft	(average random deviations)
Acoustic depth measurement	0.3 ft	(from above table)
Total Propagated Uncertainty:	0.8 ft	RMS (95%)

d. This implies that the uncertainty of the measured or cleared navigation grade is uncertain at the ± 0.8 ft (95%) confidence level. This uncertainty allowance should be factored in the tolerances used in the original studies that determine the authorized navigation depth for a project—see EM 1110-2-1613 (*Hydraulic Design of Deep-Draft Navigation Projects*). This uncertainty allowance (or tolerance) can also play in the evaluation of dredge clearance survey data and in the significant figure (rounding) resolution of recorded depths and clear grades. Figure 9-2 illustrates the uncertainty allowance estimate relative to (i.e., above and below) a nominal or required clearance grade. This uncertainty may or may not be significant on soft bottom maintenance dredging projects; however, on new work or rock-cut channels, this allowance may need to be applied to the overdepth allowance to provide additional confidence that the final channel clearance is to grade.
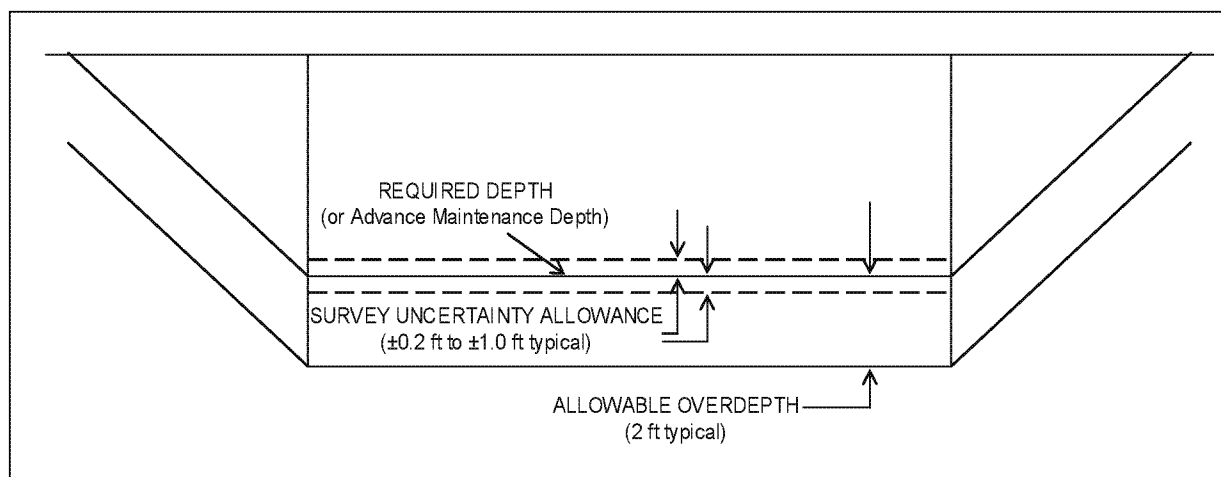

Figure 9-2. Propagated uncertainty allowance on a typical maintenance dredging template.

e. Approximate estimates of TPU in deep-draft navigation projects. Table 9-6 provides another example of general estimates for survey TPUs under nominal deep-draft project conditions, accounting for various measurement conditions largely dependent on the water surface measurement correction. These ranges may be used to estimate the TPU for a specific navigation project. Given the main variable in the table is dependent on the gage location relative to the project site (non-RTK measurements) the magnitude of this error needs to be estimated based on actual tidal range and phase conditions.

Table 9-6. Estimated TPU Allowances for Deep-Draft Navigation Projects.

Typical TPU	Water Surface Elevation Measurement Procedure	Tidal regime hydrodynamically modeled
Hard Bottom Materials		
± 0.20 foot	Determined from carrier phase GPS (RTK)	Yes
± 0.25 foot	Determined from carrier phase GPS (RTK)	No
± 0.20 foot	Estimated from gage less than 1 mile from project site	Yes
± 0.25 foot to ± 0.50 foot	Estimated from gage 1 to 5 miles from project site	No
± 0.50 foot to ± 1.0 foot	Estimated from gage > 5 miles from project site	No
± 0.50 foot to ± 2.0 foot	Estimated from gage > 10 miles from project site	No
Soft Bottom Materials (Maintenance Dredging)		
± 0.25 foot	Determined from carrier phase GPS (RTK)	Yes
± 0.25 foot to ± 1.0 foot	Estimated from gage 1 to 10 miles from project site	No
± 0.50 foot to ± 2.0 foot	Highly variable acoustic reflectivity due to suspended sediment, fluff, dense bottom vegetation, etc.	Yes

   f. *Methods for directly computing TPU of depth measurements.* A more refined estimate of the TPU in measured depths (and clearance grades) in a navigation project may be computed using algorithms developed by the Canadian Hydrographic Service (CHS) for the US Naval Oceanographic Office—see "*Error Budget Analysis for US Naval Oceanographic Office (NAVOCEANO) Hydrographic Survey Systems: Final Report for Task 2, FY 01*" (NAVOCEANO/Hare 2001). A screen capture of a TPU calculator using these algorithms is shown in Figure 9-3. This TPU calculator provides user input of the estimated accuracies of over 50 parameters making up the total (propagated) depth error budget. It is applicable to either multibeam or single-beam hydrographic systems. This calculator compares the resultant TPU with both USACE EM 1110-2-1003 (*Hydrographic Surveying*) accuracy standards and International Hydrographic Organization "*Special Publication S-44*" (IHO 1998) accuracy standards. In addition, positional errors and target detection resolutions are estimated, as shown in the figure.
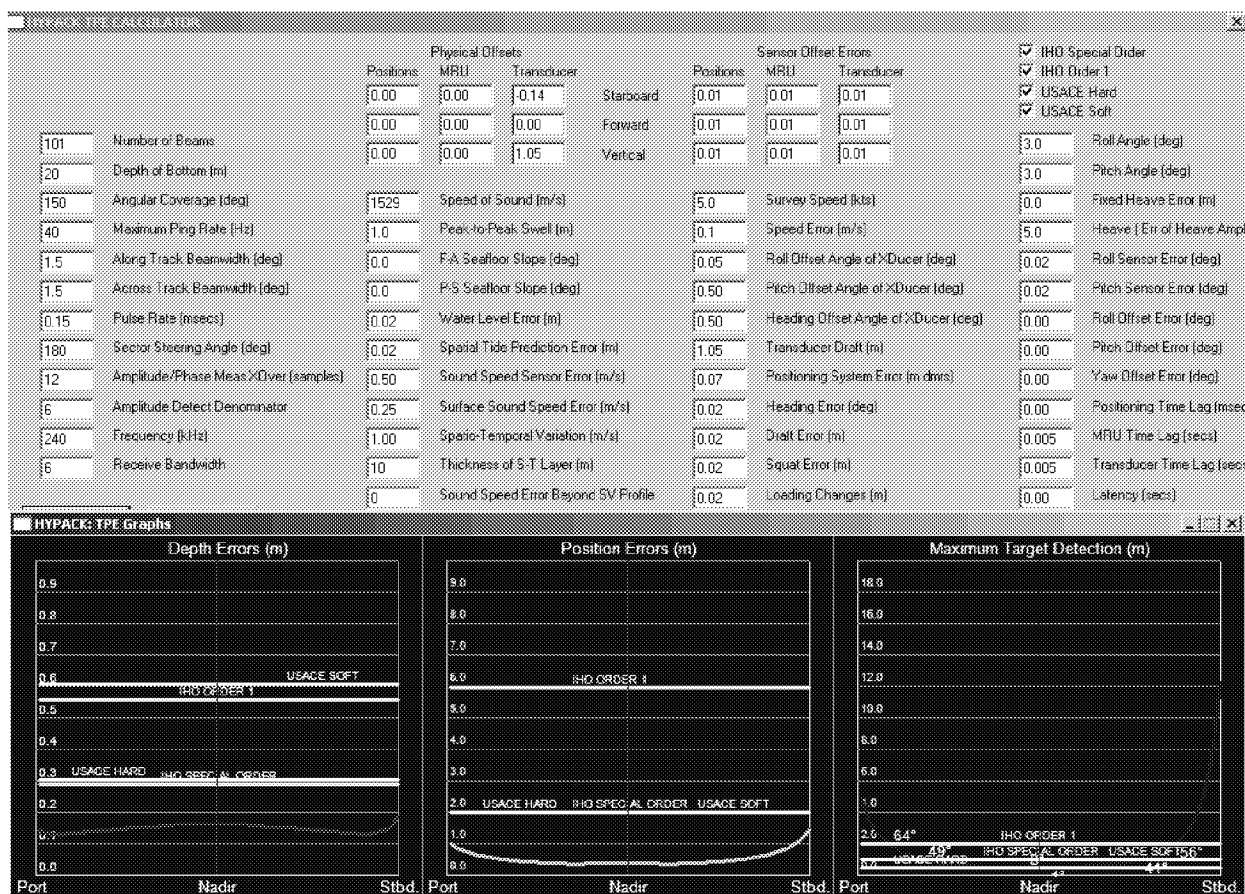
Figure 9-3. Total Propagated Uncertainty calculator for depth, position, and object detection. Values shown are for example only—users must insert estimated uncertainties for each parameter specific to their survey systems, procedures, and project. (HYPACK, Inc.)

APPENDIX A

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EM 1110-1-1003
NAVSTAR Global Positioning System Surveying

EM 1110-1-1005
Control and Topographic Surveying

EM 1110-2-1003
Hydrographic Surveying.

EM 1110-2-1009
Structural Deformation Surveying

EM 1110-2-1100
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    Part II: Physics and Gravity, SaLIS Vol. 65, No. 1, pages 5–15, March 2005.
    Part III: Height Systems, SaLIS Vol. 66, No. 2, pages 149–160, June 2006.
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EM 1110-2-1607
Tidal Hydraulics

EM 1110-2-1614
Design of Coastal Revetments, Seawalls, and Bulkheads

EM 1110-2-1913
Design and Construction of Levees

APPENDIX B

Geodetic Reference Datums and Coordinate Systems

B-1. <u>Purpose and Background</u>. This Appendix provides general background information on geodetic reference datums, coordinate systems, and local horizontal reference systems that are used to georeference USACE civil works and military construction projects. The primary focus of this appendix is on geospatial reference systems that define horizontal locations on the Earth. The use of State Plane Coordinate Systems (SPCS) is covered in detail Section II since these systems are most commonly used to reference topographic site plan surveys of local projects. Transformations between geospatial datums and coordinate systems are also discussed. Vertical datums (i.e., orthometric, tidal, hydraulic) are not included here as they were covered in Chapter 2.

    a. Most USACE site plan surveys for PED require "control surveys" to bring in a geodetic reference network to the local project site where detailed topographic surveys are performed. It is important that the correct geodetic reference network is used, and that it is consistent with the overall installation or project reference system. It is also important that these reference systems conform to the most up to date regional or nationwide reference systems (i.e., NAD83).

    b. Other topographic surveys outside Army installations or Corps civil project areas may require rigid references to established property boundaries (corner pins, section corners, road intersections/centerlines, etc.). These ties to legal boundaries and corners will thus establish the reference system by which all topographic survey features are detailed. Regional geodetic networks may or may not be required on such surveys, depending on local practice or statute.

SECTION I

Geodetic Reference Systems

B-2. <u>General</u>. The discipline of surveying consists of locating points of interest on the surface of the earth. The positions of points of interest are defined by coordinate values that are referenced to a predefined mathematical surface. In geodetic surveying, this mathematical surface is called a datum, and the position of a point with respect to the datum is defined by its coordinates. The reference surface for a system of control points is specified by its position with respect to the earth and its size and shape. Control points are points with known relative positions tied together in a network. Densification of the network refers to adding more fixed control points to the network. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network. Reference systems can be based on the geoid, ellipsoid, or a plane. The earth's gravitational force can be modeled to create a positioning reference frame that rotates with the earth. The geoid is such a surface (an equipotential surface of the earth's gravity field) that best approximates MSL. The orientation of this surface at a given point on geoid is defined by the plumb line. The plumb line is oriented tangent to the local gravity vector. Surveying instruments can be readily oriented with respect to the gravity field because its physical forces can be sensed with simple mechanical devices.

B-3. <u>Geodetic Coordinates</u>. A coordinate system is defined by the location of the origin, orientation of its axes, and the parameters (coordinate components) which define the position of a point within the coordinate system. Terrestrial coordinate systems are widely used to define the position of points on the terrain because they are fixed to the earth and rotate with it. The origin of terrestrial systems can be specified as either geocentric (i.e., origin at the center of the earth, such as NAD83) or topocentric (i.e., origin at a point on the surface of the earth, such as NAD27). The orientation of terrestrial coordinate systems is described with respect to its poles, planes, and axes.

a. Geocentric coordinates. Geocentric coordinates have an origin at the center of the earth, as shown in Figure B-1. GPS coordinates are initially observed on this type of reference system. For example, a coordinate on such a system might be displayed on a GPS receiver as:

$$X = \quad 668400.506 \text{ m}$$
$$Y = -4929214.152 \text{ m}$$
$$Z = \quad 3978967.747 \text{ m}$$

GPS receivers will transform these geocentric coordinates into a geographic coordinate system described below.
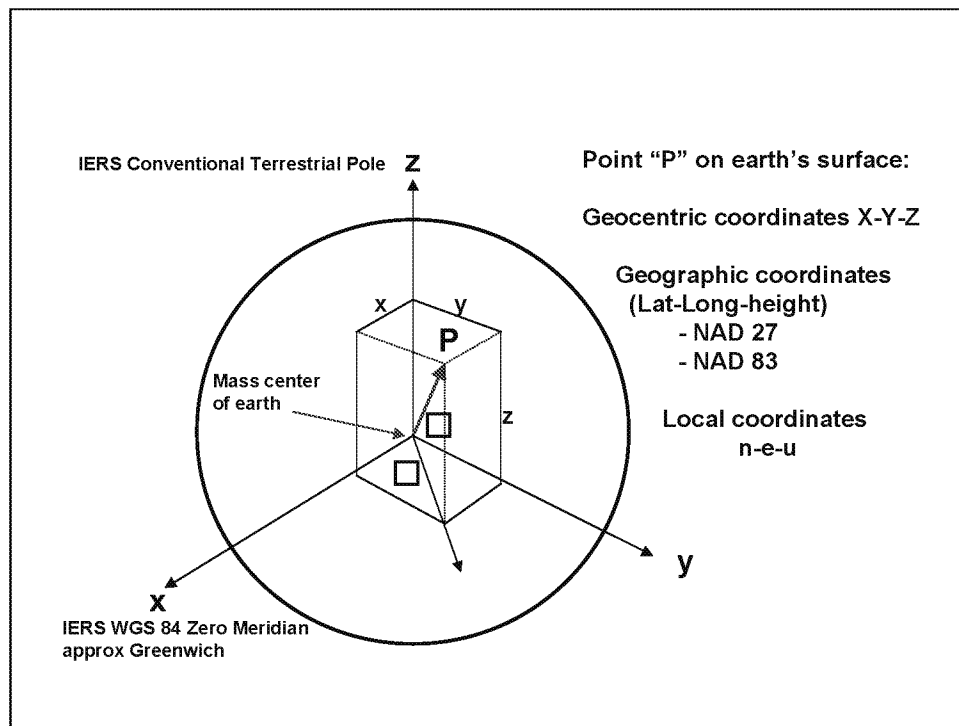
Figure B-1. Earth-centered earth-fixed coordinate reference frames.

b. Geodetic or Geographic coordinates. Geographic coordinate components consist of latitude (ϕ), longitude (λ), and ellipsoid height (h). Geodetic latitude, longitude, and ellipsoid height define the position of a point on the surface of the Earth with respect to some "reference ellipsoid." The most common reference ellipsoid used today is the WGS84, which will be described in more detail in a later section.

(1) Geodetic latitude (ϕ). The geodetic latitude of a point is the acute angular distance between the equatorial plane and the normal through the point on the ellipsoid measured in the meridian plane (Figure B-1). Geodetic latitude is positive north of the equator and negative south of the equator.

(2) Geodetic longitude (λ). The geodetic longitude is the angle measured counter-clockwise (east), in the equatorial plane, starting from the prime meridian (Greenwich meridian), to the meridian of the defined point (Figure B-1). In the continental United States, longitude is commonly reported as a west longitude. To convert easterly to westerly referenced longitudes, the easterly longitude must be subtracted from 360 degrees as shown below.

East-West Longitude Conversion

$$\lambda \, (W) = [\, 360 - \lambda \, (E) \,]$$

For example:

$$\lambda\,(E) = 282^{\,d}\,52^{\,m}\,36.345^{\,s}\,E$$
$$\lambda\,(W) = [\,360^{\,d} - 282^{\,d}\,52^{\,m}\,36.345^{\,s}\,E\,]$$
$$\lambda\,(W) = 77^{\,d}\,07^{\,m}\,23.655^{\,s}\,W$$

(3) Ellipsoid Height (h). The ellipsoid height is the linear distance above the reference ellipsoid measured along the ellipsoidal normal to the point in question. The ellipsoid height is positive if the reference ellipsoid is below the topographic surface and negative if the ellipsoid is above the topographic surface.

(4) Geoid Separation (N). The geoid separation (or often termed "geoidal height") is the distance between the reference ellipsoid surface and the geoid surface measured along the ellipsoid normal. The geoid separation is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid.

(5) Orthometric Height (H). The orthometric height is the vertical distance of a point above or below the geoid.

(6) The relationships between the ellipsoid height geoid height, and the orthometric height were illustrated in Chapter 2.

B-4. Datums. A datum is a coordinate surface used as reference for positioning control points. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network.

a. Geodetic datum. Five parameters are required to define an ellipsoid-based datum. The semi-major axis (a) and flattening (f) define the size and shape of the reference ellipsoid; the latitude and longitude of an initial point; and a defined azimuth from the initial point define its orientation with respect to the earth. The NAD27 and NAD83 systems are examples of horizontal geodetic datums. Such a reference surface is developed from an ellipsoid of revolution that best approximates the geoid. An ellipsoid of revolution provides a well-defined mathematical surface to calculate geodetic distances, azimuths, and coordinates.

b. Horizontal datum. A horizontal datum is defined by specifying (1) the geometric surface (plane, ellipsoid, sphere) used in coordinate, distance, and directional calculations, (2) the initial reference point (origin), and (3) a defined orientation, azimuth or bearing from the initial point. The "horizontal datum" for most topographic surveys is usually defined relative to the fixed control points (monuments and/or bench marks) that were used to control the individual shots. These "control points" may, in turn, be referenced to a local installation/compound control network and/or to a national NSRS CORS station.

c. Project datum. A project datum is defined relative to local control and might not be directly referenced to a geodetic datum. Project datums are usually defined by a system with perpendicular axes, and with arbitrary coordinates for the initial point, and with one (principal)

axis oriented toward an assumed north. A chainage-offset system may also be used as a reference, with the PIs (points of intersection) either marked points or referenced to some other coordinate system.

d. Vertical datum. A vertical datum is a reference system used for reporting elevations. The two most common nationwide systems are the NGVD29 and the NAVD88. See Chapter 2 for details on these orthometric datums.

e. The National Spatial Reference System (NSRS). The NSRS is that component of the National Spatial Data Infrastructure (NSDI) that contains all geodetic control contained in the NGS database. (See Chapter 2 for details on the NSRS).

B-5. WGS84 Reference Ellipsoid. The GPS satellites are referenced to the WGS84 ellipsoid. The origin of the WGS84 Cartesian system is the earth's center of mass, as shown in Figure B-1. The Z-axis is parallel to the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by the Bureau International Heure (BIH), and equal to the rotation axis of the WGS84 ellipsoid. The X-axis is the intersection of the WGS84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS84 ellipsoid. The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 degrees east of the X-axis and equal to the Y-axis of the WGS84 ellipsoid. The DOD continuously monitors the origin, scale, and orientation of the WGS84 reference frame and references satellite orbit coordinates to this frame. Updates are shown as WGS84 (GXXX), where "XXX" refers to a GPS week number starting on 29 September 1996.

a. It is a common misconception that the resultant position of a GPS survey is referenced to WGS84. While this would be the case if we were using GPS in an absolute mode (no reference/base station), in the differential GPS mode, the geospatial coordinates have been shifted from the WGS84 ellipsoid to the GRS80 ellipsoid when the reference receiver is using NAD83 coordinates.

b. Over the years there have been several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying and mapping community. Table B-1 lists just a few of these reference systems along with their mathematical defining parameters. Note that GRS80 is the actual reference ellipsoid for NAD83; however, the difference in the axis between GRS80 and WGS84 ellipsoids is insignificant but the origins differ by over 2 meters. Transformation techniques are used to convert between different datums and coordinate systems. Most GPS software has built in transformation algorithms for the more common datums.

Table B-1. Reference Ellipsoids and Related Coordinate Systems.

Reference Ellipsoid	Coordinate System (Datum/Frame)	Semimajor axis (meters)	Shape (1/flattening)
Clarke 1866	NAD27	6378206.4	1/294.9786982
WGS72	WGS72	6378135	1/298.26
GRS80	NAD83 (XX)	6378137	1/298.257222101
WGS84	WGS84 (GXXX)	6378137	1/298.257223563
ITRS	ITRF (XX)	6378136.49	1/298.25645

SECTION II

Horizontal Coordinate Systems

B-6. <u>General</u>. Geocentric, geographic, or geodetic coordinates described above are rarely used to reference site plan topographic surveys or maps. Engineering site plan drawings are normally referenced to a local SPCS, or in some cases, a metric-based UTM system. They may also be referenced to an arbitrary coordinate system relative to some point on the project--a monument, property corner, road intersection, etc. Most construction drawings also contain "chainage-offset" (stationing) reference systems. In most cases, control surveys performed for setting project control will be computed and adjusted using the SPCS. The following paragraphs describe horizontal coordinate systems commonly used on facility site plan mapping and related control surveys.

B-7. <u>Geographic coordinates</u>. The use of geographic coordinates as a system of reference is accepted worldwide. It is based on the expression of position by latitude (parallels) and longitude (meridians) in terms of arc (degrees, minutes, and seconds) referred to the equator (north and south) and a prime meridian (east and west). The degree of accuracy of a geographic reference (GEOREF) is influenced by the map scale and the accuracy requirements for plotting and scaling. Examples of GEOREFs are as follows:

40° N 132° E (referenced to degrees of latitude and longitude).
40°21' N 132°14' E (referenced to minutes of latitude and longitude).
40°21'12" N 132°14'18" E (referenced to seconds of latitude and longitude).
40°21'12.4" N 132°14'17.7" E (referenced to tenths of seconds of latitude and longitude).
40°21'12.45" N 132°14'17.73" E (referenced to hundredths of seconds of latitude and longitude).

US military maps and charts include a graticule (parallels and meridians) for plotting and scaling geographic coordinates. Graticule values are shown in the map margin. On maps and charts at scales of 1:250,000 and larger, the graticule may be indicated in the map interior by lines or ticks at prescribed intervals (for example, scale ticks and interval labeling at the corners of 1:50,000 at 1minute [in degrees, minutes, and seconds] and again every 5 minutes).

B-8. <u>Horizontal Datums and Reference Frames</u>. The following paragraphs briefly describe the most common datums used to reference CONUS projects.

a. North American Datum of 1927 (NAD27). NAD27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations. NAD27 is a best fit for the continental United States. The fixed datum reference point is located at Meades Ranch, Kansas. The longitude origin of NAD27 is the Greenwich Meridian with a south azimuth orientation. The original network adjustment used 25,000 stations. The relative precision between initial point monuments of NAD27 is by definition 1:100,000, but coordinates on any given monument in the network contain errors of varying degrees. As a result, relative accuracies between points on NAD27 may be far less than the nominal 1:100,000. The reference units for NAD27 are US Survey Feet. This datum is no longer supported by NGS, and USACE

commands have been gradually transforming their project coordinates over to the NAD83 described below. Approximate conversions of points on NAD27 to NAD83 may be performed using CORPSCON, a transformation program developed by ERDC/TEC. Since NAD27 contains errors approaching 10 m, transforming highly accurate GPS observations to this antiquated reference system is not the best approach.

b. North American Datum of 1983 (NAD83). The nationwide horizontal reference network was redefined in 1983 and readjusted in 1986 by the NGS. It is known as the North American Datum of 1983, Adjustment of 1986, and is referred to as NAD83 (1986). (Subsequent adjustments have been made). NAD83 used far more stations (250,000) and observations than did NAD27, including a few satellite-derived coordinates, to readjust the national network. The longitude origin of NAD83 is the Greenwich Meridian with a north azimuth orientation. The fixed adjustment of NAD83 (1986) has an average precision of 1:300,000. NAD83 is based upon GRS80, an earth-centered reference ellipsoid which for most, but not all, practical purposes is equivalent to WGS84. With increasingly more accurate uses of GPS, the errors and misalignments in NAD83 (1986) became more obvious (they approached 1 meter), and subsequent refinements outlined below have been made to correct these inconsistencies.

c. High Accuracy Reference Networks (HARN). (Figure B-2). Within a few years after 1986, more refined GPS measurements had allowed geodesists to locate the earth's center of mass with a precision of a few centimeters. In doing so, these technologies revealed that the center of mass that was adopted for NAD83 (1986) is displaced by about 2 m from the true geocenter. These discrepancies caused significant concern as the use of highly accurate GPS measurements proliferated. Starting with Tennessee in 1989, each state--in collaboration with NGS and various other institutions--used GPS technology to establish regional reference frames that were to be consistent with NAD83. The corresponding networks of GPS control points were originally called High Precision Geodetic Networks (HPGN). Currently, they are referred to as High Accuracy Reference Networks (HARN). This latter name reflects the fact that relative accuracies among HARN control points are better than 1 ppm, whereas relative accuracies among pre-existing control points were nominally only 10 ppm. Positional differences between NAD83 (1986) and NAD83 (HARN) can approach 1 meter.
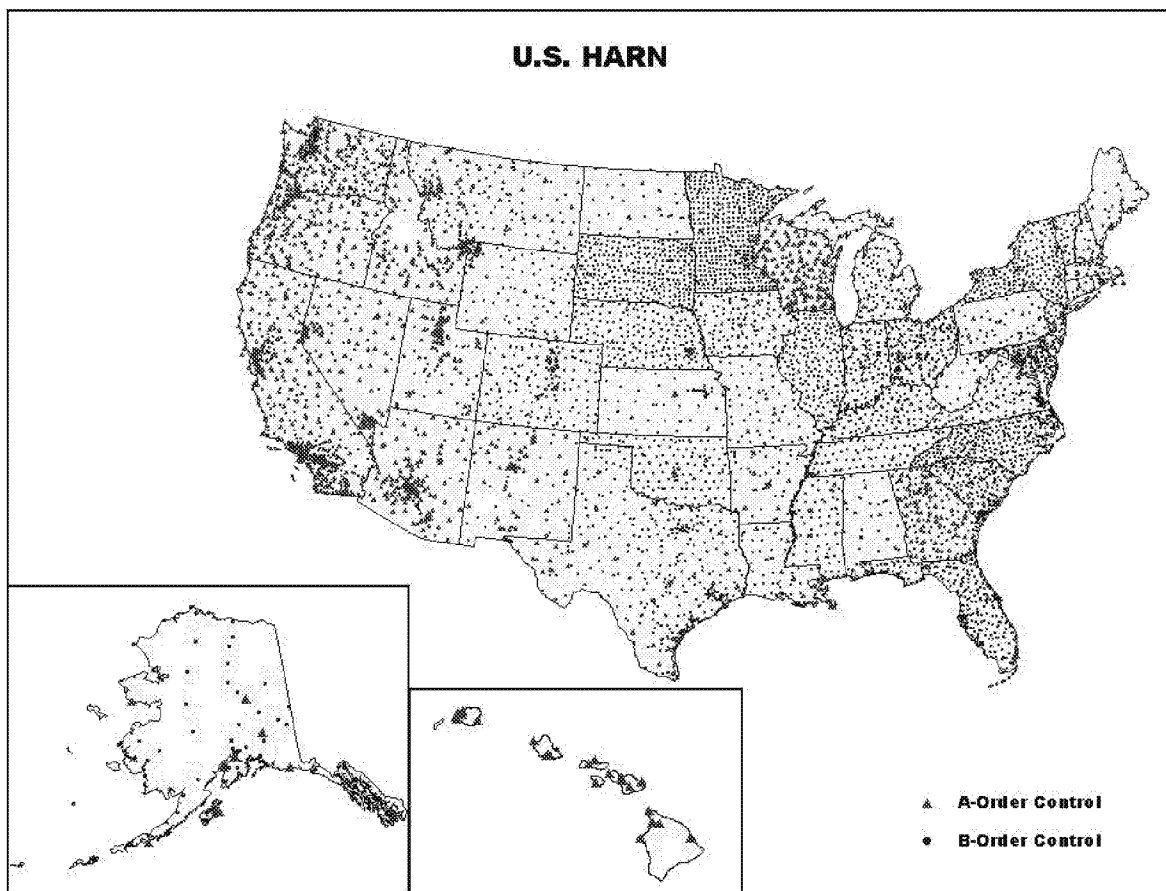
Figure B-2. High Accuracy Reference Network control points.

d. Continuously Operating Reference Stations (CORS). The regional HARNs were subsequently further refined (or "realized") by NGS into a network of Continuously Operating Reference Stations, or CORS. This CORS network was additionally incorporated with the International Terrestrial Reference System (ITRS), i.e. the ITRF. CORS are located at fixed points throughout CONUS and at some OCONUS points--see Figure B-3. This network of high-accuracy points can provide GPS users with centimeter level accuracy where adequate CORS coverage exists. Coordinates of CORS stations are designated by the year of the reference frame, e.g., NAD83 (CORS 96). Positional differences between NAD83 (HARN) and NAD83 (CORS) are less than 10 cm. More importantly, positional difference between two NAD83 (CORSxx) points is typically less than 2 cm. Thus, GPS connections to CORS stations will be of the highest order of accuracy. USACE commands can easily connect and adjust GPS-observed points directly with CORS stations using a number of methods, including the NGS on-line program OPUS (see Chapter 3 and EM 1110-1-1003). CORS are particularly useful when precise control is required in a remote area, from which a topographic survey may be performed. With only 1 to 2 hours of static DGPS observations, reference points can often be established to an ellipsoid accuracy better than ± 0.25 ft in X-Y-Z.
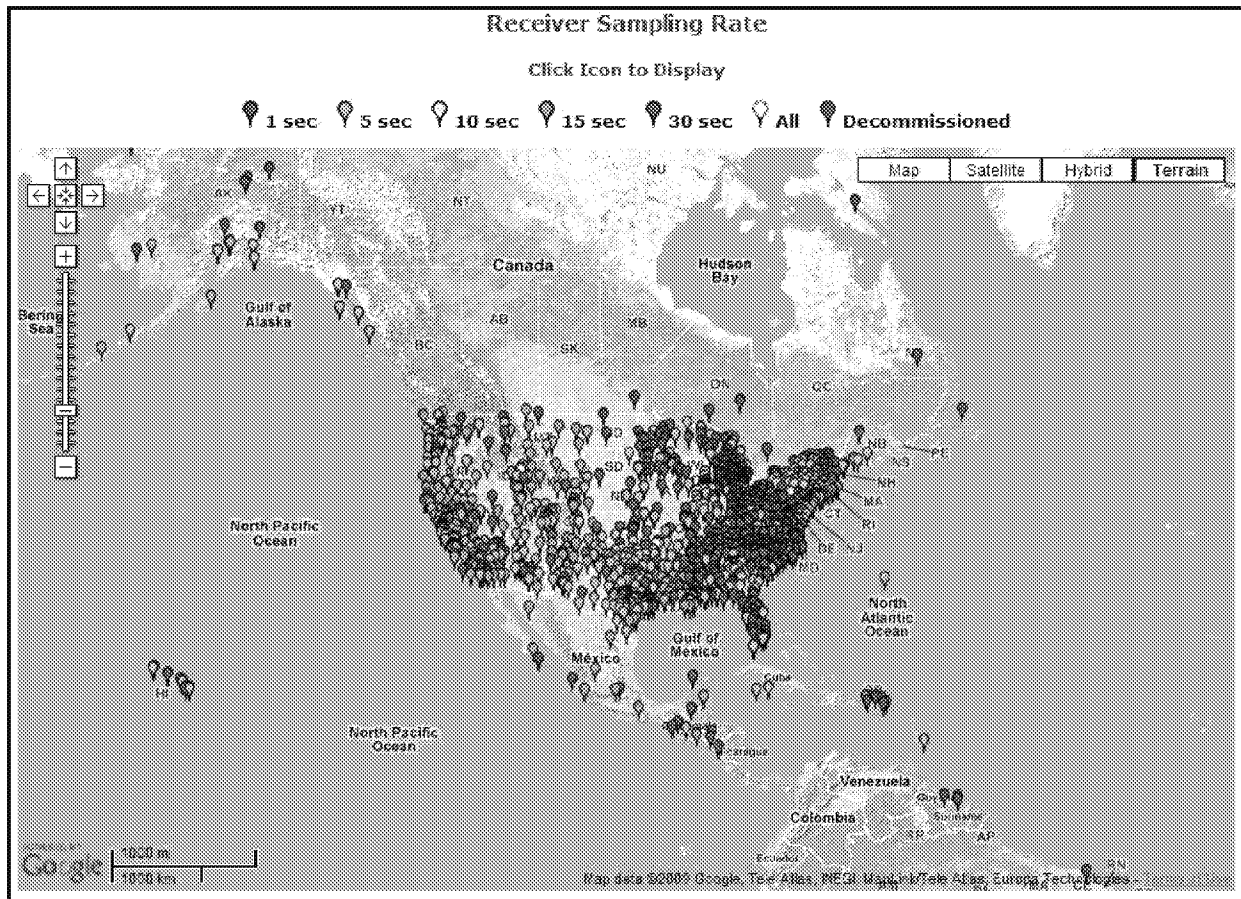
Figure B-3. Continuously Operating Reference Stations as of 2010. (NGS)

e. 2007 National Readjustment. (See Figure B-4). A readjustment of NSRS was completed in 2007 by the NGS. The adjustment was undertaken to resolve inconsistencies between the existing statewide HARNs, the Federal Base Network (FBN) adjustments, and the nationwide CORS system, as well as between states. Individual local and network accuracy estimates were also derived from this effort. This readjustment includes ~70,000 passive geodetic control monuments constrained to the NAD83 (CORS96) realization. NAD83 (NSRS2007) was created by adjusting GPS data collected during various geodetic surveys performed between the mid-1980's and 2005. For this adjustment NAD83 (CORS96) positional coordinates for ~700 CORS were held fixed. The CORS 2002 epoch was used for all states except AZ, OR, WA, CA, and AK where an epoch of 2007 was used. Derived NAD83 (NSRS2007) positional coordinates should be consistent with corresponding NAD83 (CORS96) positional coordinates to within the accuracy of the GPS data used in the adjustment.
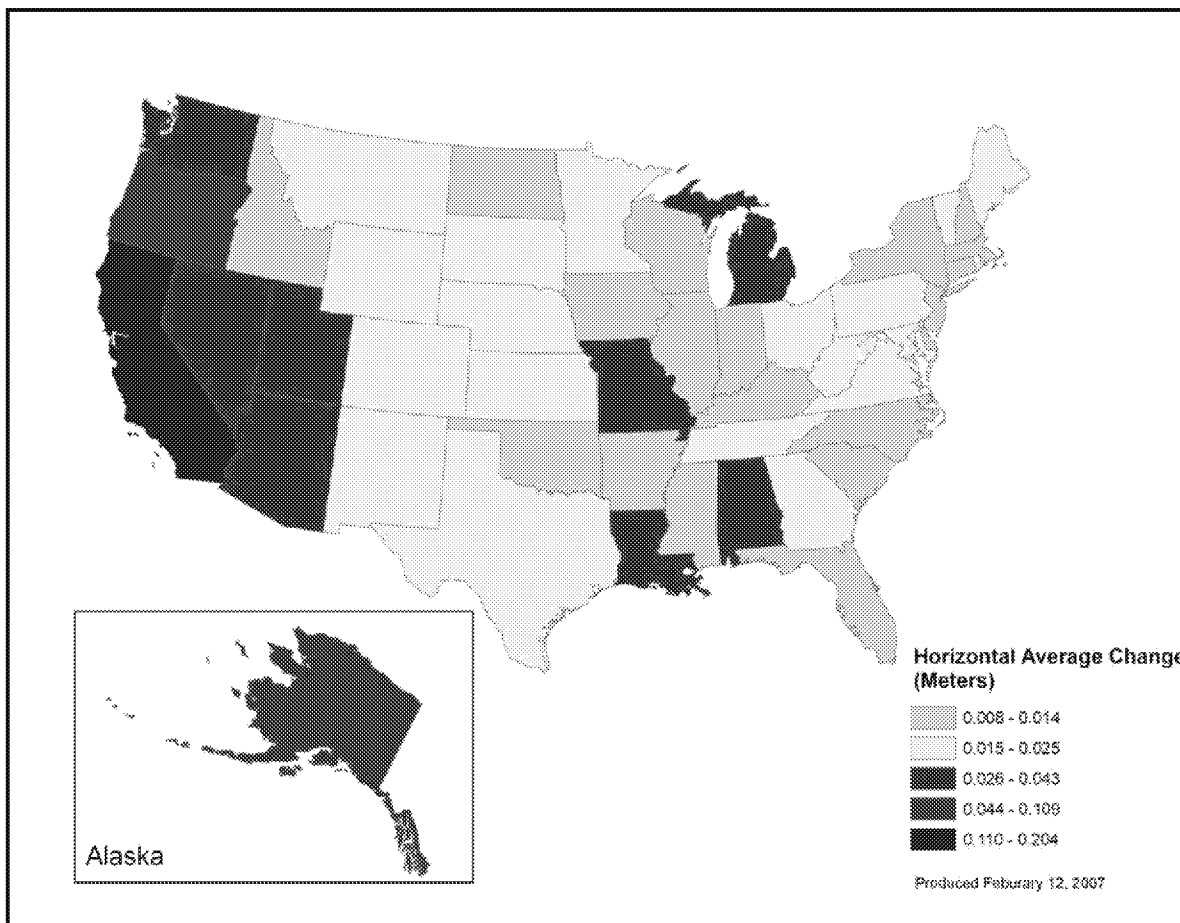
Figure B-4. Horizontal coordinate shifts between NAD83 and NAD83 in meters (NSRS 2007).

    f. International Terrestrial Reference Frame (ITRF). The ITRF is a highly accurate geocentric reference frame with an origin at the mass center of the earth. The ITRF is continuously monitored and updated by the International Earth Rotation Service (IERS) using very-long-baseline-interferometry (VLBI) and other techniques. These observations allow for the determination of small movements of fixed points on the earth's surface due to crustal motion, rotational variances, tectonic plate movement, etc. These movements can average 10 to 20 mm/year in CONUS, and may become significant when geodetic control is established from remote reference stations. These refinements can be used to accurately determine GPS positions observed on the basic WGS84 reference frame. NAD83 coordinates are defined based on the ITRF year/epoch in which it is defined, e.g., ITRF 89, ITRF 96, ITRF 2000. For highly accurate positioning where plate velocities may be significant, users should use the same coordinate reference frame and epoch for both the satellite orbits and the terrestrial reference frame. USACE requirements for these precisions on control surveys would be rare, and would never be applicable to local facility mapping surveys. Those obtaining coordinates from NGS datasheets must take care not to use ITRF values. The relationship between ITRF, NAD83, and the geoid is illustrated in Figure B-5.
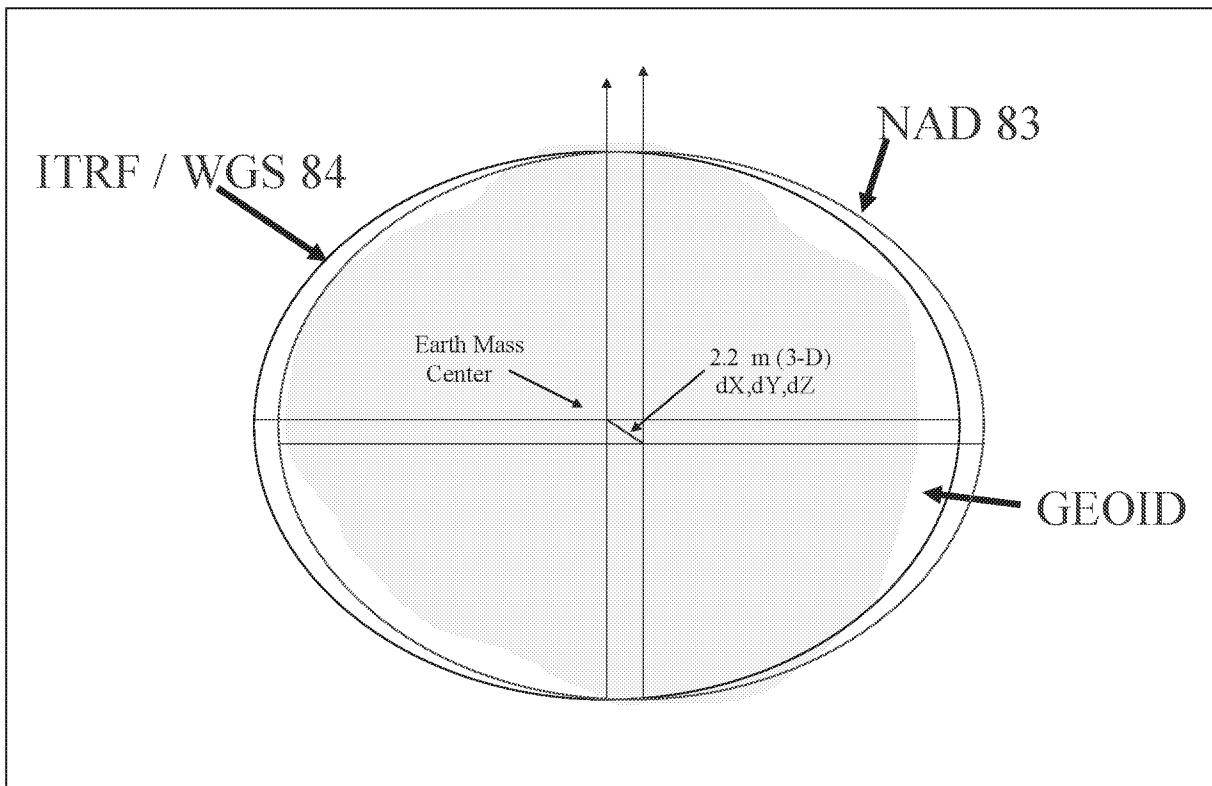
Figure B-5. Relationship between ITRF, NAD83, and the geoid.

B-9. State Plane Coordinate Systems.

    a. General. State Plane Coordinate Systems (SPCS) were developed by the NGS to provide plane coordinates over a limited region of the earth's surface. To properly relate geodetic coordinates (ϕ-λ-h) of a point to a 2D plane coordinate representation (Northing, Easting), a conformal mapping projection must be used. Conformal projections have mathematical properties that preserve differentially small shapes and angular relationships to minimize the errors in the transformation from the ellipsoid to the mapping plane. Map projections that are most commonly used for large regions are based on either a conic or a cylindrical mapping surface (Figure B-6). The projection of choice is dependent on the north-south or east-west areal extent of the region. Typically states with limited east-west dimensions and indefinite north-south extent use the Transverse Mercator (TM) type projection while states with limited north-south dimensions and indefinite east-west extent use the Lambert projection. The SPCS is designed to minimize the spatial distortion at a given point to approximately one part in ten thousand (1:10,000). To satisfy this criterion, the SPCS has been divided into zones that have a maximum width or height of approximately one hundred and fifty eight statute miles (158 miles). Therefore, each state may have several zones and/or may employ both the Lambert (conic) and Transverse Mercator (cylindrical) projections. The projection state plane coordinates are referenced to a specific geodetic datum (i.e. the datum that the initial geodetic coordinates are referenced to must be known).

LAMBERT CONFORMAL
CONIC PROJECTION
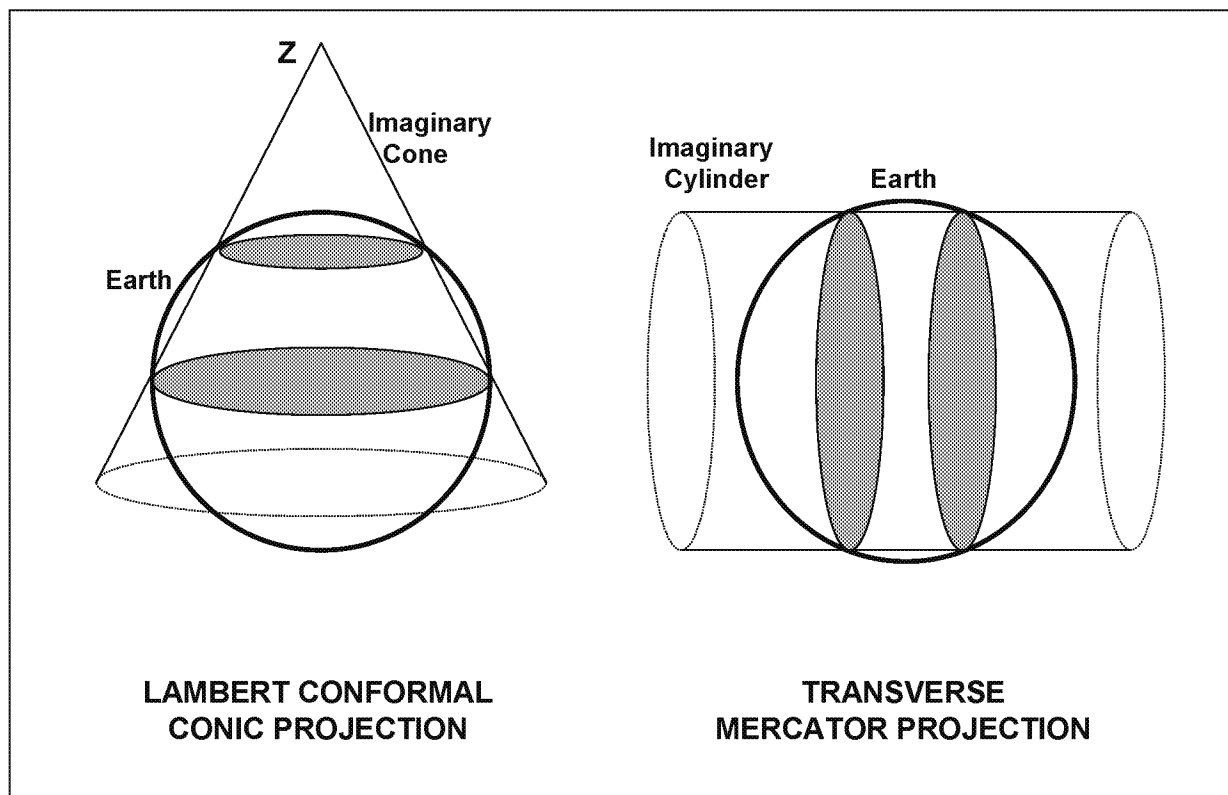
TRANSVERSE
MERCATOR PROJECTION

Figure B-6. Common map projections.

   b. Transverse Mercator (TM). The Transverse Mercator projection uses a cylindrical surface to cover limited zones on either side of a central reference longitude. Its primary axis is rotated perpendicular to the symmetry axis of the reference ellipsoid. Thus, the TM projection surface intersects the ellipsoid along two lines equidistant from the designated central meridian longitude (Figure B-7). Distortions in the TM projection increase predominantly in the east-west direction. The scale factor for the Transverse Mercator projection is 1.0000 where the cylinder intersects the ellipsoid. The scale factor is less than one between the lines of intersection, and greater than one outside the lines of intersection. The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid. To compute the state plane coordinates of a point, the latitude and longitude of the point and the projection parameters for a particular TM zone or state must be known.
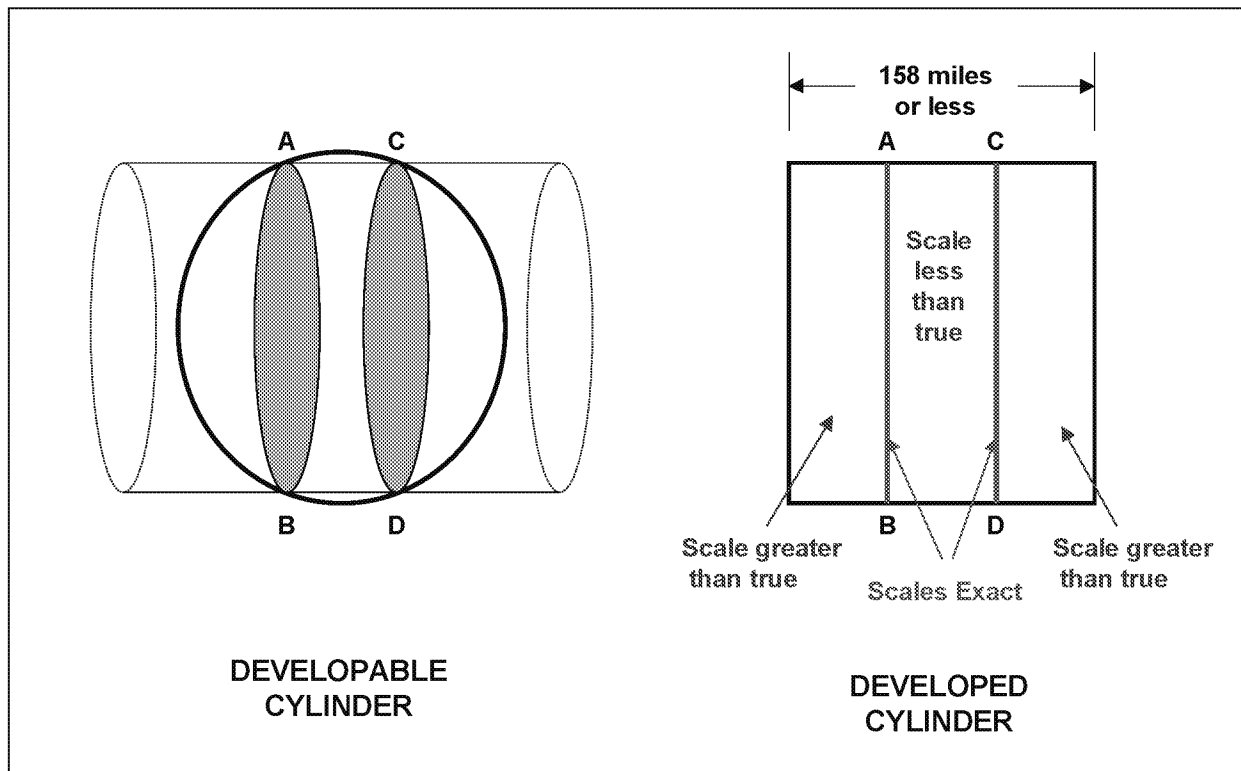
DEVELOPABLE CYLINDER

DEVELOPED CYLINDER

158 miles or less

A   C

Scale less than true

B   D

Scale greater than true

Scales Exact

Scale greater than true

Figure B-7. Transverse Mercator Projection.

Vertex

R

Projection Limits < 158 miles

Standard Parallels scale exact
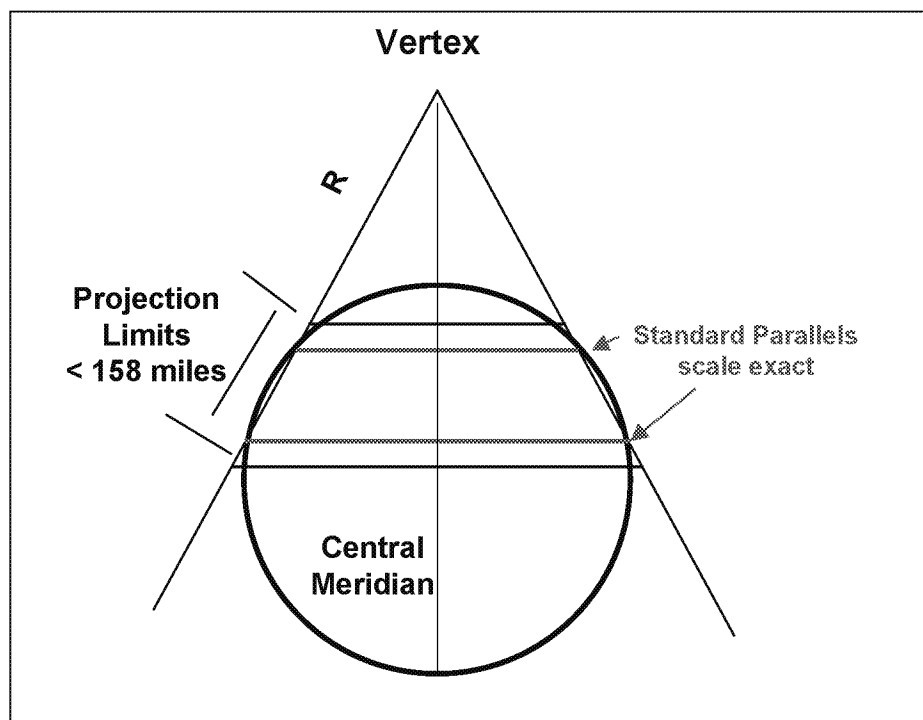
Central Meridian

Figure B-8. Lambert Conformal Conic Projection.

c. Lambert Conformal Conic (LCC). The Lambert projection uses a conic surface to cover limited zones of latitude adjacent to two parallels of latitude. Its primary axis is coincident with the symmetry axis of the reference ellipsoid. Thus, the LCC projection intersects the ellipsoid along two standard parallels (Figure B-8). Distortions in the LCC projection increase predominantly in the north-south direction. The scale factor for the Lambert projection is equal to 1.0000 at each standard parallel and is less than one inside, and greater than one outside the standard parallels. The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid and remains constant along the standard parallels.

d. SPCS zones. Figure B-9 depicts the various SPCS zones in the US. The unique state zone number provides a standard reference when using transformation software developed by NGS and USACE. The state zone number remains constant in both NAD27 and NAD83 coordinate systems. There have been some changes in the number of zones in a few of the states, for example, California dropped zone 0407 which is now included in zone 0405 and Montana went from three zones to one.

Figure B-9. SPCS zones identification numbers for the various states.

e. Scale units. State plane coordinates can be expressed in both feet and meters. State plane coordinates defined on the NAD27 datum are published in feet. State plane coordinates defined on the NAD83 datum are published in meters; however, state and federal agencies can request the NGS to provide coordinates in feet. If NAD83 based state plane coordinates are defined in meters and the user intends to convert those values to feet, the proper meter-feet conversion factor (shown below) must be used. Some states use the International Survey Foot rather than the US Survey Foot in the conversion of feet to meters (see Figure B-10).

International Survey Foot:	*1 International Foot = 0.3048 meter (exact)*
US Survey Foot:	*1 US Survey Foot = 1200 / 3937 meter (exact)*

The use of the incorrect conversion factor can lead to significant errors in the resultant coordinates.

Figure B-10. English-metric conversions in the various states (NGS).

B-10. <u>Grid Elevations, Scale Factors, and Convergence</u>. In all planer grid systems, the grid projection only approximates the ellipsoid (or roughly the ground), and "ground-grid" corrections must be made for measured distances or angles (directions). Measured ground distances must be corrected for (1) elevation (sea level factor), and (2) ground to grid plane (scale factor). Figure B-11 illustrates a reduction of a measured distance (D) down to the ellipsoid distance (S). Not shown is the subsequent reduction from the ellipsoid length to a grid system length. Observed directions (or angles) must also be corrected for grid convergence. Also shown on the figure is the relationship between ellipsoid heights (h), geoid heights (N), and orthometric heights (H).

$$S = D \times \left(\frac{R}{R+h} \right)$$

$$h = N + H$$

$$S = D \times \left[\frac{R}{R+N+H} \right]$$

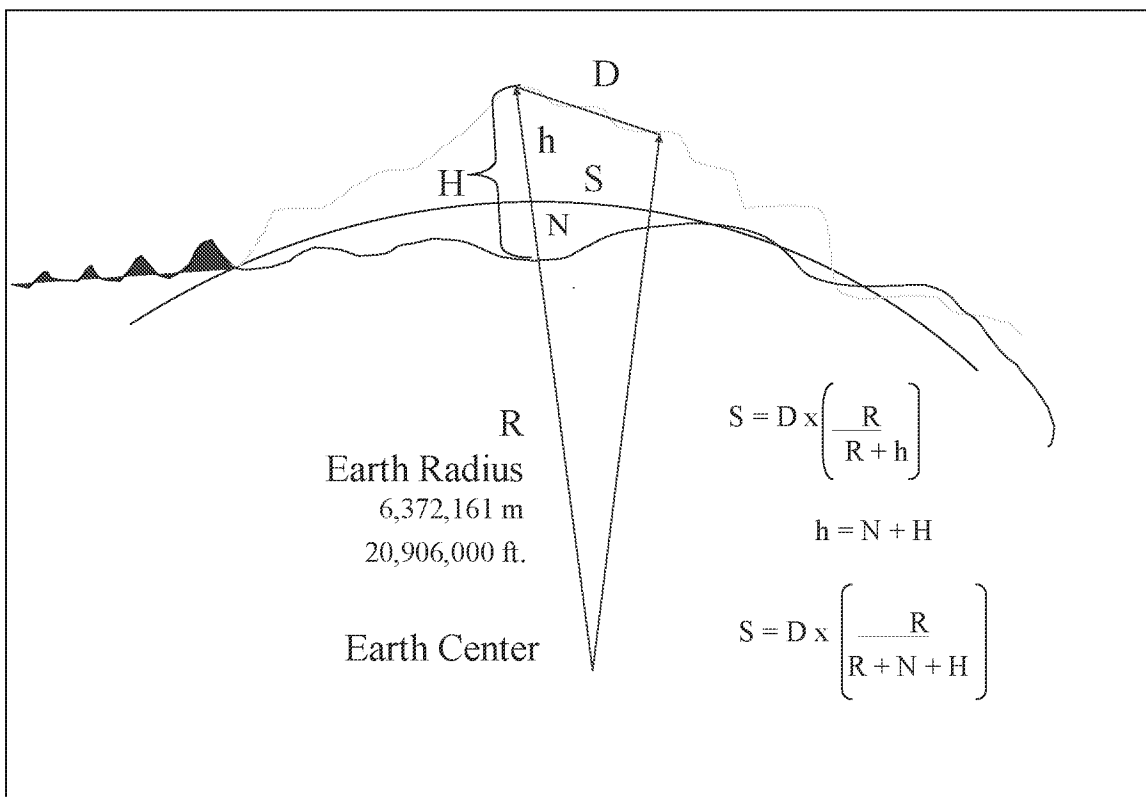R
Earth Radius
6,372,161 m
20,906,000 ft.

Earth Center

Figure B-11. Reduction of measured slope distance D to ellipsoid distance S (NGS).

a. Grid factor. For most topographic surveys covering a small geographical site, these two factors can be combined into a constant "grid factor" or "combined scale factor."

Grid factor = Sea Level Factor x Scale Factor

then: *Ground Distance = Grid Distance / Grid Factor*

or

Grid Distance = Ground Distance x Grid Factor

b. Convergence. Between two fixed points, the geodetic azimuth will differ from the grid azimuth. This difference is known as "convergence" and varies with the distance from the central meridian of the projection. Thus, if a geodetic azimuth is given between two fixed points (inversed from published geographic coordinates, astronomic, or GPS), then it must be corrected for convergence to obtain an equivalent grid azimuth. If lengthy control traverses are being computed on a SPCS or UTM grid, then additional second term corrections to observed angles may be required--e.g., the "t-T" correction used in older survey manuals.

c. Use of data collectors. The above grid corrections should rarely have to be performed when modern survey data collectors are being used. These total station or RTK data collectors (with full COGO and adjustment capabilities) will automatically perform all the necessary

geographic to grid coordinate translations, including sea level reductions and local grid system conversions that are later transformed and adjusted into an established SPCS grid at a true elevation.

d. References. Many DA publications (i.e., Field Manuals) and surveying textbooks contain information, procedures, and examples of these grid transforms.

B-11. Universal Transverse Mercator Coordinate System. Universal Transverse Mercator (UTM) coordinates are used in surveying and mapping when the size of the project extends through several state plane zones or projections. UTM coordinates are also utilized by the DOD for tactical mapping, charting, and geodetic applications. It may also be used to reference site plan engineering surveys if so requested in CONUS or OCONUS installations. The UTM projection differs from the TM projection in the scale at the central meridian, origin, and unit representation. The scale at the central meridian of the UTM projection is 0.9996. In the Northern Hemisphere, the northing coordinate has an origin of zero at the equator. In the Southern Hemisphere, the southing coordinate has an origin of 10,000,000 m. The easting coordinate has an origin of 00,000 m) at the central meridian. The UTM system is divided into 60 longitudinal zones. Each zone is 6 degrees in width extending 3 degrees on each side of the central meridian. UTM coordinates are always expressed in meters. USACE program CORPSCON can be used to transform coordinates between UTM and SPCS systems. Additional details on UTM grids and survey computations thereon may be found in DA publications.

B-12. The US Military Grid-Reference System (FM 3-34.331). The US Military Grid-Reference System (MGRS) is designed for use with UTM grids. For convenience, the earth is generally divided into 6° by 8° geographic areas, each of which is given a unique grid-zone designation. These areas are covered by a pattern of 100,000-meter squares. Two letters (called the 100,000-meter-square letter identification) identify each square. This identification is unique within the area covered by the grid-zone designation.

a. The MGRS is an alphanumeric version of a numerical UTM grid coordinate. Thus, for that portion of the world where the UTM grid is specified (80° south to 84° north), the UTM grid-zone number is the first element of a military grid reference. This number sets the zone longitude limits. The next element is a letter that designates a latitude bond. Beginning at 80° south and proceeding northward, 20 bands are lettered C through X. In the UTM portion of the MGRS, the first three characters designate one of the areas within the zone dimensions.

b. A reference that is keyed to a gridded map (of any scale) is made by giving the 100,000-meter-square letter identification together with the numerical location. Numerical references within the 100,000-meter square are given to the desired accuracy in terms of the easting and northing grid coordinates for the point.

c. The final MGRS position coordinate consists of a group of letters and numbers that include the following elements:

(1) The grid-zone designation.

(2) The 100,000-meter-square letter identification.

(3) The grid coordinates (also referred to as rectangular coordinates) of the numerical portion of the reference, expressed to a desired refinement.

(4) The reference is written as an entity without spaces, parentheses, dashes, or decimal points.

d. Examples of MGRS coordinates are as follows:

> *18S (locating a point within the grid-zone designation).*
> *18SUU (locating a point within a 100,000-meter square).*
> *18SUU80 (locating a point within a 10,000-meter square).*
> *18SUU8401 (locating a point within a 1,000-meter square).*
> *18SUU836014 (locating a point within a 100-meter square).*

e. To satisfy special needs, a reference can be given to a 10-meter square and a 1-meter square, as shown below.

> *18SUU83630143 (locating a point within a 10-meter square).*
> *18SUU8362601432 (locating a point within a 1-meter square).*

f. There is no zone number in the polar regions. A single letter designates the semicircular area and the hemisphere. The letters A, B, Y, and Z are used only in the polar regions. An effort is being made to reduce the complexity of grid reference systems by standardizing a single, worldwide grid reference system.

B-13. <u>US National Grid System</u>. A US National Grid (USNG) system has been developed to improve public safety, commerce, and aid the casual GPS user with an easy to use geoaddress system for identifying and determining location with the help of a USNG gridded map and/or a USNG enabled GPS system. The USNG can provide for whatever level of precision is desired. Many users may prefer to continue using the UTM format for applications requiring precision greater than 1 meter.

a. Grid Zone Designation (GZD). The US geographic area is divided into 6-degree longitudinal zones designated by a number and 8-degree latitudinal bands designated by a letter. Each area is given a unique alphanumeric Grid Zone Designator--e.g., 18S.

b. 100,000-meter square identification. Each GZD 6x8 degree area is covered by a specific scheme of 100,000-meter squares where each square is identified by two unique letters-- e.g., 18SUJ identifies a specific 100,000-meter square in the specified GZD.

c. Grid coordinates. A point position within the 100,000-meter square shall be given by the UTM grid coordinates in terms of its Easting (E) and Northing (N). An equal number of digits shall be used for E and N where the number of digits depends on the precision desired in position referencing. In this convention, the reading shall be from left with Easting first and then Northing, for example:

> *18SUJ20 - Locates a point with a precision of 10 km*
> *18SUJ2306 - Locates a point with a precision of 1 km*
> *18SUJ234064 - Locates a point with a precision of 100 meters*
> *18SUJ23480647 - Locates a point with a precision of 10 meters*
> *18SUJ2348306479 - Locates a point with a precision of 1 meter*

The number of digits in Easting and Northing can vary, depending on specific requirements or application.

B-14. Chainage-Offset Coordinate Systems. Most linear engineering and construction projects (roads, railways, canals, navigation channels, levees, floodwalls, beach renourishment, etc.) are locally referenced using the traditional engineering chainage-offset system (Figure B-12). Usually, SPCS coordinates are provided at the PIs, from which, given the alignment between PIs, a SPCS coordinate can then be computed for any given station-offset point. Chainage-offset systems are used for locating cross-sections along even centerline stations. Topographic elevation and feature data is then collected along each section relative to the centerline. Likewise, road, canal, and levee alignments can be staked out relative to station-offset parameters, and internally in a total station or RTK system data collector, these offsets may actually be transformed from a SPCS.

a. Stationing. Alignment stationing (or chainage) zero references are arbitrarily established for a given project or sectional area. For example, stationing on a navigation project usually commences offshore on coastal projects and runs inland or upstream. Stationing follows the channel centerline alignment. Stationing may be accumulated through each PI or zero out at each PI or new channel reach. Separate stationing is established for widener sections, turning basins, levees, floodwalls, etc. Each district may have its own convention. Stationing coordinates use "+" signs to separate the second- and third-place units (XX + XX.XX). Metric chainage often separates the third and fourth places (XXX + XXX.XX) to distinguish the units from English feet; however, some districts use this convention for English stationing units.

b. Offsets. Offset coordinates are distances from the centerline alignment of a road, levee, or navigation channel. Offsets carry plus/minus coordinate values. Normally, offsets are positive to the right (looking toward increasing stationing). Some USACE Districts designate cardinal compass points (east-west or north-south) in lieu of a coordinate sign. On some navigation projects, the offset coordinate is termed a "range," and is defined relative to the project centerline or, in some instances, the channel-slope intersection line (toe). Channel or canal offsets may be defined relative to a fixed baseline on the bank or levee.
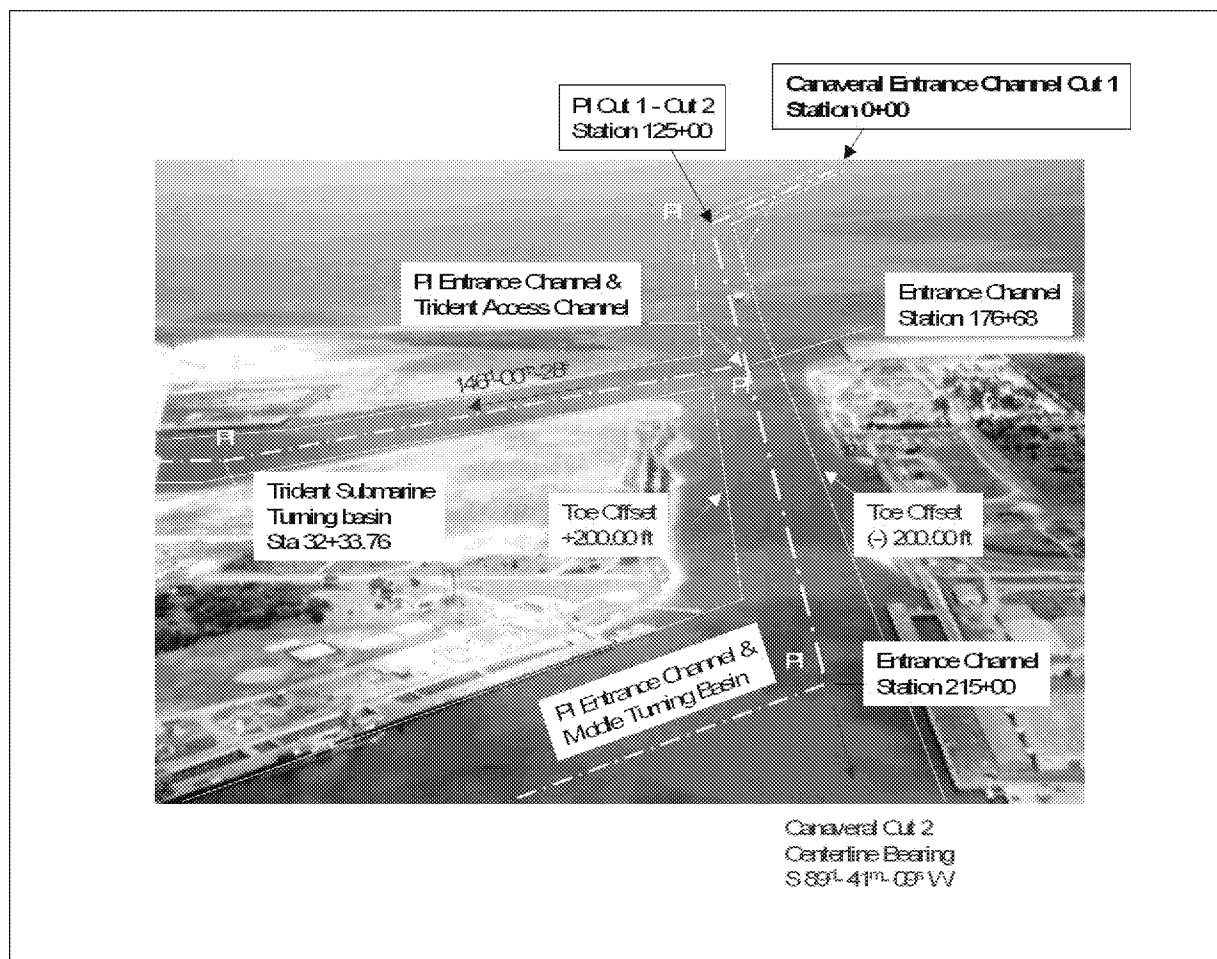
Figure B-12. Chainage-offset project control scheme for a typical deep-draft navigation project--
Cape Canaveral, FL. (Jacksonville District)

c. Azimuth. Azimuths are computed relative to the two defining PIs. Either 360-deg azimuth or bearing designations may be used. Azimuths should be shown to the nearest second.

d. Other local alignments. Different station-offset reference grids may be established for individual portions of a project. River sections and coastal beach sections are often aligned perpendicular to the project/coast. Each of these sections is basically a separate local datum with a different reference point and azimuth alignment. Beach sections may also be referenced to an established coastal construction setback line. Circular and transition (spiral) curve alignments are also found in some rivers, canals, and flood risk management projects such as spillways and levees. Surveys will generally be aligned to the chainage and offsets along such curves. Along inland waterways, such as the Mississippi River, stationing is often referenced to either arbitrary or monumented baselines along the bank. In many instances, a reference baseline for a levee is used, and surveys for revetment design and construction are performed from offsets to this line. Separate baselines may exist over the same section of river, often from levees on opposite banks or as the result of revised river flow alignments. Baseline stationing may increase either upstream or downstream. Most often, the mouth of a river is considered the starting point

(Station 0 + 00), or the river reaches are summed to assign a station number at the channel confluence. Stationing may increase consecutively through PIs or reinitialize at channel turns. In addition, supplemental horizontal reference may also be made to a river mile designation system. River mile systems established years ago may no longer be exact if the river course has subsequently realigned itself. River mile designations can be used to specify geographical features and provide navigation reference for users.

B-15. Datum Conversions and Transformation Methods.

a. Topographic site plan surveys of a project can be performed on any coordinate system. Many localized total station topographic surveys are initiated on (or referenced to) an arbitrary coordinate grid system, e.g., X=5,000 ft, Y=5,000 ft, Z=100 ft, and often elevation or scale reductions are ignored. Planimetric and topographic data points collected on this arbitrary grid in a data collector are then later translated, rotated, scaled, and/or "best fit" to some established geographical reference system--e.g., the local SPCS.

b. The process of converting the observed topographic points on the arbitrary grid system to an established geographical reference system (e.g., NSRS/SPCS) is termed a "datum transformation." In order to perform this transformation, a few points (preferably three or more) in the topographic database must be referenced to the external reference system. These "control" points on a topographic survey have been previously established relative to an installation or project's primary control network. They normally were established using more accurate "geodetic control" survey procedures, such as differential leveling, static or kinematic DGPS observations, or total station traverse.

c. CORSPCON. Federal Geodetic Control Subcommittee (FGCS) members, which includes USACE, have adopted NAD83 as the standard horizontal datum for surveying and mapping activities performed or funded by the Federal government. To the extent practicable, legally allowable, and feasible, USACE should use NAD83 in its surveying and mapping activities. Transformations between NAD27 coordinates and NAD83 coordinates are generally obtained using the "CORPS Convert" (CORPSCON) software package or other North American Datum Conversion (e.g., NADCON) based programs.

d. Conversion techniques. USACE survey control published in the NGS control point database has been already converted to NAD83 values. However, most USACE survey control was not originally in the NGS database and was not included in the NGS readjustment and redefinition of the national geodetic network. Therefore, USACE will have to convert this control to NAD83. Coordinate conversion methods considered applicable to USACE projects are discussed below.

(1) Resurvey from NAD83 Control. A new survey using NGS published NAD83 control could be performed over the entire project. This could be either a newly authorized project or one undergoing major renovation or maintenance. Resurvey of an existing project must tie into all monumented points. Although this is not a datum transformation technique, and would not normally be economically justified unless major renovation work is being performed, it can be used if existing NAD27 control is of low density or accuracy.

(2) Readjustment of Survey. If the original project control survey was connected to NGS control stations, the survey may be readjusted using the NAD83 coordinates instead of the NAD27 coordinates originally used. This method involves locating the original field notes and observations, and completely readjusting the survey and fixing the published NAD83 control coordinates.

(3) Mathematical Transformations. Since neither of the above methods can be economically justified on most USACE projects, mathematical approximation techniques for transforming project control data to NAD83 have been developed. These methods yield results which are normally within ± 1 ft of the actual values and the distribution of errors are usually consistent within a local project area. Since these coordinate transformation techniques involve approximations, they should be used with caution when real property demarcation points and precise surveying projects are involved. When mathematical transformations are employed they should be adequately noted so that users will be aware of the conversion method.

e. Horizontal datum transformation methods. Coordinate transformations from one geodetic reference system to another can be most practically made either by using a local seven-parameter transformation or by interpolation of datum shift values across a given region.

(1) Seven parameter transformations. For worldwide (OCONUS) and local datum transformations, many surveying textbooks contain additional information, procedures, and examples and may be consulted.

(2) Grid-shift transformations. Current methods for interpolation of datum shift values use the difference between known coordinates of common points from both the NAD27 and NAD83 adjustments to model a best-fit shift in the regions surrounding common points. A grid of approximate datum shift values is established based on the computed shift values at common points in the geodetic network. The datum shift values of an unknown point within a given grid square are interpolated along each axis to compute an approximate shift value between NAD27 and NAD83. Any point that has been converted by such a transformation method should be considered as having only approximate NAD83 coordinates.

(3) NADCON/CORPSCON. NGS developed the transformation program NADCON, which yields consistent NAD27 to NAD83 coordinate transformation results over a regional area. This technique is based on the above grid-shift interpolation approximation. NADCON was reconfigured into a more comprehensive program called CORPSCON. Technical documentation and operating instructions for CORPSCON can be obtained at the AGC web site listed in Chapter 1. This software converts between the following coordinate systems:

NAD27	NAD83	SPCS 27	SPCS 83
UTM 27	UTM 83	NGVD29	NAVD88
GEOID03/09	HARN		

Since the overall CORPSCON datum shift (from point to point) varies throughout North America, the amount of datum shift across a local project is also not constant. The variation can

be as much as 0.1 ft per mile. Examples of some 27 to NAD83 based coordinate shift variations that can be expected over a 10,000-ft section of a project are shown below:

Project Area	SPCS Reference	Per 10,000 feet
Baltimore, MD	1900	0.16 ft
Los Angeles, CA	0405	0.15 ft
Mississippi Gulf Coast	2301	0.08 ft
Mississippi River (IL)	1202	0.12 ft
New Orleans, LA	1702	0.22 ft
Norfolk, VA	4502	0.08 ft
San Francisco, CA	0402	0.12 ft
Savannah, GA	1001	0.12 ft
Seattle, WA	4601	0.10 ft

(4) Scale changes. The above scale changes will cause project alignment data to distort by unequal amounts. Thus, a 10,000-ft tangent on NAD27 project coordinates could end up as 9,999.91 feet after mathematical transformation to NAD83 coordinates. Although such differences may not be appear significant from a lower-order construction survey standpoint, the potential for such errors must be recognized. Therefore, the transformations will not only significantly change absolute coordinates on a project and the datum transformation process will slightly modify the project's design dimensions and/or construction orientation and scale. For example, on a navigation project, an 800.00-ft wide channel could vary from 799.98 to 800.04 feet along its reach. This variation could also affect grid alignment azimuths. Moreover, if the local SPCS 83 grid was further modified, then even larger dimension changes can result. Correcting for distortions may require recomputation of coordinates after conversion to ensure original project dimensions and alignment data remain intact. This is particularly important for property and boundary surveys. A less accurate alternative is to compute a fixed shift to be applied to all data points over a limited area.

(5) Maximum shift limits. Determining the maximum area over which such a fixed shift can be applied is important. Computing a fixed conversion factor with CORPSCON can be made to within ±1 foot. Typically, this fixed conversion would be computed at the center of a sheet or at the center of a project and the conversions in X and Y from NAD27 to NAD83 and from SPCS 27 to SPCS 83 indicated by notes on the sheets or data sets. Since the conversion is not constant over a given area, the fixed conversion amounts must be explained in the note. The magnitude of the conversion factor change across a sheet is a function of location and the drawing scale. Whether the magnitude of the distortion is significant depends on the nature of the project. For example, a 0.5-ft variation on an offshore navigation project may be acceptable for converting depth sounding locations, whereas a 0.1-ft change may be intolerable for construction layout on an installation. In any event, the magnitude of this gradient should be computed by CORPSCON at each end (or corners) of a sheet or project. If the conversion factor variation exceeds the allowable tolerances, then a fixed conversion factor should not be used. Two examples of determining a fixed conversion factor are illustrated below.

Example 1. Assume a 1 inch = 40 ft scale site plan map on existing SPCS 27 (VA South Zone 4502). Using CORPSCON, convert existing SPCS 27 coordinates at the sheet center and corners to SPCS 83 (US Survey Foot), and compare SPCS 83-27 differences.

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
Center	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
of Sheet	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
NW	N 3,527,595.553	Y 246,700.000	dY = 3,280,895.553
Corner	E 11,920,522.693	X 2,437,525.000	dX = 9,482,997.693
NE	N 3,527,595.556	Y 246,700.000	dY = 3,280,895.556
Corner	E 11,921,522.691	X 2,438,525.000	dX = 9,482,997.691
SE	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,921,522.702	X 2,438,525.000	dX = 9,482,997.702
SW	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,920,522.704	X 2,437,525.000	dX = 9,482,997.704

(6) Since coordinate differences do not exceed 0.03 feet in either the X or Y direction, the computed SPCS 83-27 coordinate differences at the center of the sheet may be used as a fixed conversion factor to be applied to all existing SPCS 27 coordinates on this drawing.

Example 2. Assuming a 1 inch = 1,000 ft base map is prepared of the same general area, a standard drawing will cover some 30,000 feet in an east-west direction. Computing SPCS 83-27 differences along this alignment yields the following:

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
West	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
End	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
East	N 3,527,095.364	Y 246,200.000	dY = 3,280,895.364
End	E 11,951,022.104	X 2,468,025.000	dX = 9,482,997.104

(7) The conversion factor gradient across this sheet is about 0.2 ft in Y and 0.6 ft in X. Such small changes are not significant at the plot scale of 1 inch = 1,000 ft; however, for referencing basic design or construction control, applying a fixed shift across an area of this size is not recommended -- individual points should be transformed separately. If this 30,000-ft distance were a navigation project, then a fixed conversion factor computed at the center of the sheet would suffice for all bathymetric features. Caution should be exercised when converting portions of projects or military installations or projects that are adjacent to other projects that may not be converted. If the same monumented control points are used for several projects or parts of the same project, different datums for the two projects or parts thereof could lead to surveying and mapping errors, misalignment at the junctions and layout problems during construction.

B-25

f. Dual grids ticks. Depicting both NAD27 and NAD83 grid ticks and coordinate systems on maps and drawings should be avoided where possible. This is often confusing and can increase the chance for errors during design and construction. However, where use of dual grid ticks and coordinate systems is unavoidable, only secondary grid ticks in the margins leads to less confusion.

g. Field survey methods. If GPS is used to set new control points referenced to higher order control many miles from the project (e.g., CORS networks), inconsistent data may result at the project site. If the new control is near older control points that have been converted to NAD83 using CORPSCON, two slightly different network solutions can result, even though both have NAD83 coordinates. In order to avoid these situations, it is recommended that all project control (old and new) be tied into the same reference system--preferably the NSRS.

h. Local project datums. Local project datums that are not referenced to NAD27 cannot be mathematically converted to NAD83 with CORPSCON. Field surveys connecting them to other stations that are referenced to NAD83 are required.

B-16. Horizontal Transition Plan from NAD27 to NAD83.

a. General. Not all USACE maps, engineering site drawings, documents, and associated products containing coordinate information will require conversion NAD27 to NAD83. To insure an orderly and timely transition to NAD83 is achieved for the appropriate products, the following general guidelines should be followed:

(1) Initial surveys. All initial surveys should be referenced to NAD83.

(2) Active projects. Active projects where maps, site drawings or coordinate information are provided to non-USACE users (e.g., NOAA, USCG, FEMA, and others in the public and private sector) coordinates should be converted to NAD83 the next time the project is surveyed or maps or site drawings are updated for other reasons.

(3) Inactive projects. For inactive projects or active projects where maps, site drawings or coordinate information are not normally provided to non-USACE users, conversion to NAD83 is optional.

b. Levels of effort. For maps and site drawings the conversion process entails one of three levels of effort:

(1) Conversion of coordinates of all mapped details to NAD83, and redrawing the map,

(2) Replace the existing map grid with a NAD83 grid,

(3) Simply adding a datum note.

For surveyed points, control stations, alignment, and other coordinated information, conversion must be made either through a mathematical transformation or through readjustment of survey observations.

APPENDIX C

Requirements and Procedures for Referencing Coastal Navigation Projects to
Mean Lower Low Water (MLLW) Datum

C-1. Purpose. This appendix is an edited reprint of USACE technical guidance that was issued in 1993 to implement applicable portions of Section 224 of the Water Resources Development Act of 1992 (WRDA 1992). This guidance was originally issued as an Engineer Technical Letter—i.e., ETL 1110-2-349, which was subsequently rescinded. Much of the guidance in this ETL is still applicable to those Corps projects that have not been fully converted to the latest federal reference datum or tidal epoch. This includes technical considerations and general implementation procedures for referencing coastal navigation projects to a consistent Mean Lower Low Water (MLLW) datum based on tidal characteristics defined and published by the US Department of Commerce. References herein to the "NOS" (the National Ocean Service) now apply to the current NOAA organization responsible for tides and water levels—the "Center for Operational Oceanographic Products and Services" (CO-OPS).

C-2. Applicability. The technical guidance in this ETL [Appendix] applies to commands having responsibilities for design of river and harbor navigation projects on the Atlantic, Gulf, and Pacific coasts, and where such projects are subject to tidal influence.

C-3. References. [*Outdated references in the original ETL were deleted*]

    a. Rivers and Harbors Appropriation Act of 1915 (38 Stat. 1053; 33 U.S.C. 562).

    b. Water Resources Development Act of 1992 (WRDA 92), Section 224, Channel Depths and Dimensions.

    c. The National Tidal Datum Convention of 1980, US Department of Commerce.

C-4. Background.

    a. Depths of USACE navigation projects in coastal areas subject to tidal influences are currently referred to a variety of vertical reference planes, or datums. Most project depths are referenced to a local or regional datum based on tidal phase criteria, such as Mean Low Water, Mean Lower Low Water, Mean Low Gulf, Gulf Coast Low Water Datum, etc. Some of these tidal reference planes were originally derived from US Department of Commerce, National Ocean Service (NOS) observations and definitions used for the various coasts. Others were specifically developed for a local project and may be without reference to an established vertical network (e.g., National Geodetic Vertical Datum of 1929) or a tidal reference. Depending on the year of project authorization, tidal epoch, procedures, and the agency that established or connected to the reference datum, the current adequacy of the vertical reference may be uncertain, or in some cases, unknown. In some instances, project tidal reference grades may not

have been updated since original construction. In addition, long-term physical effects may have significantly impacted presumed relationships to the NOS MLLW datum.

b. The National Tidal Datum Convention of 1980 established one uniform, continuous tidal datum for all marine waters of the United States, its territories, and Puerto Rico. This convention thereby lowered the reference plane (and tidal definition) of both the Atlantic and Gulf coasts from a mean low water datum to a MLLW datum. In addition, the National Tidal Datum Epoch (NTDE) was updated to the 1960-1978 period and mean higher/high water datums used for legal shoreline delineation were redefined. The latest tidal epoch update is the 1983-2001 period.

c. Since 1989, nautical charts published by NOAA reference depths (or soundings) to the local MLLW reference datum, also termed a "chart datum." US Coast Guard (USCG) Notices to Mariners also refer depths or clearances over obstructions to MLLW. Depths and clearances reported on USACE project/channel condition surveys provided to NOAA, for incorporation into their published charts in plan or tabular format, must be on the same NOS MLLW reference as the local chart of the project site.

d. WRDA 92, Section 224, requires consistency between USACE project datums and NOAA marine charting datums. This act amended Section 5 of the Rivers and Harbors Appropriation Act of 1915 to define project depths of operational projects as being measured relative to a MLLW reference datum for all coastal regions. Only the Pacific coast was previously referenced to MLLW. The amendment states that this reference datum shall be as defined by the Department of Commerce for nautical charts and tidal prediction tables for a given area. This provision requires USACE project reference grades be consistent with NOS MLLW (latest epoch).

C-5. Impact of MLLW Definition on USACE Projects.

a. Corps navigation projects that are referenced to older datums (e.g., Mean Low Water along the Atlantic coast or various Gulf coast low water reference planes) must be converted to and correlated with the local MLLW tidal reference established by the NOS. Changes in project grades due to redefining the datum from mean low water to NOS MLLW will normally be small, and in many cases will be compensated for by offsetting secular sea level or epochal increases occurring over the years. Thus, impacts on dredging due to the redefinition of the datum reference are expected to be small and offsetting in most cases.

b. All Corps project reference datums, including those currently believed to be on MLLW, must be checked to insure that they are properly referred to the latest tidal epoch, and that variations in secular sea level, local reference gage or bench mark subsidence/uplift, and other long-term physical phenomena are properly accounted for. In addition, projects should be reviewed to insure that tidal phase and range characteristics are properly modeled and corrected during dredging, surveying, and other marine construction activity, and that specified project clearances above grade properly compensate for any tidal range variances. Depending on the age and technical adequacy of the existing MLLW reference (relative to NOS MLLW), significant differences could be encountered. Such differences may dictate changes in channels

currently maintained. Future NOS tidal epoch revisions after the current 1983-2001 period will also change the project reference planes.

c. Conversion of project datum reference to NOS MLLW may or may not involve field tidal observations. In many projects, existing NOS tidal records can be used to perform the conversion, and short-term simultaneous tidal comparisons will not be required. Tidal observations and/or comparisons will be necessary for projects in areas not monitored by NOS or in cases where no recent or reliable observations are available.

C-6. Implementation Actions. A number of options are available to USACE commands in assessing individual projects for consistency and accuracy of reference datums, and performing the necessary tidal observations and/or computations required to adequately define NOS MLLW project reference grades. Datum establishment or verification may be done using USACE technical personnel, through an outside Architect-Engineer contract, by another Corps district or laboratory having special expertise in tidal work, or through reimbursable agreement with NOS. Regardless of who performs the tidal study, all work should be closely coordinated with both the USC&GS [now NGS] and NOS [CO-OPS] in the Department of Commerce.

a. Technical specifications. The general techniques for evaluating, establishing, and/or transferring a tidal reference plane are fully described in the USACE and Department of Commerce publications referenced in paragraph C-3. These references should be cited in technical specifications used for a tidal study contract or reimbursable agreement with another agency/command.

b. Department of Commerce contacts. Before and during the course of any tidal study, close coordination is required with the NOS.

c. Sources. If in-house forces are not used, the following outside sources may be utilized to perform a tidal study of a project, including any field tidal observations.

(1) Architect-Engineer (A-E) Contract. A number of private firms possess capabilities to perform this work. Either a fixed-scope contract or indefinite delivery contract form may be utilized. In some instances, this type of work may be within the scope of existing contracts. Contact NOS to obtain a typical technical specification which may be used in developing a scope of work. The references in paragraph C-3 of this appendix must be cited in the technical scope of work for the A-E contract.

(2) Reimbursable Support Agreement. Tidal studies and datum determinations may be obtained directly from the NOS, Department of Commerce, via a reimbursable support agreement. A cooperative agreement can be configured to include any number of projects within a district. Funds are provided to NOS by standard inter-agency transfer methods and may be broken down to individual projects. Contact the NOS to coordinate and schedule a study agreement.

d. Scheduling of conversions. Section 224 of WRDA 92 did not specify an implementation schedule for converting existing projects to NOS MLLW (or verifying the

adequacy of an existing MLLW datum). It is recommended that a tidal datum study be initiated during a project's next major maintenance cycle.

e. Funding. No centralized account has been established to cover the cost of converting projects to NOS MLLW datum. Project Operations and Maintenance funds will be used to cover the cost of tidal studies and/or conversions on existing projects. For new construction, adequate funding should be programmed during the initial planning and study phases. Budget estimates for performing the work can be obtained from NOS.

f. MLLW relationship to national vertical network. USACE tidal bench marks should be connected to the NSRS--currently NAVD88. Project condition surveys, maps, reports, studies, etc. shall clearly depict the local relationship between MLLW datum and the NSRS vertical network.

g. Changes in dredging. It is not expected that the datum conversion will significantly impact dredging requirements. USACE commands should request HQUSACE guidance should a datum conversion cause a significant change in a channel's maintained depth.

APPENDIX D

Tampa Harbor Navigation Project: Evaluation of the Project Datum and Implementation of a
VDatum Model (Jacksonville District)

D-1. Purpose. This appendix contains excerpts from Jacksonville District reports that illustrate the evaluation of the adequacy of a project datum for a typical deep-draft navigation project. It outlines the procedures for updating the reference tidal datum along with procedures for implementing use of VDatum for dredging and construction surveys.

    a. Section 1. Section 1 in this appendix contains excerpts from a 2007 Comprehensive Evaluation of Project Datums (CEPD) report on Tampa Harbor. This report was prepared by HQUSACE directive. This CEPD report evaluated the current condition of the project's datums and recommended corrective actions to bring the project into compliance with Corps policy.

    b. Section 2. Section 2 outlines excerpts from a 2009 internal Jacksonville District channel framework report on subsequent actions proposed to correct the deficiencies identified in the Section 1 CEPD report. It also illustrates recommended VDatum site calibration requirements for a project with full VDatum and partial RTN coverage. (Portions of this report were revised and edited since it was based on a superseded version of VDatum and the latest [2010] release of the VDatum model for Tampa Bay has not yet been field calibrated).

D-2. Project Description. The total project consists of a channel from the Gulf of Mexico to ports in Tampa Bay—see Figure D-1. Project features include the entrance channel from the Gulf of Mexico to Hillsborough Bay. At Hillsborough Bay, the channel splits into two legs, with one continuing west to Port Tampa and the other east to Gadsden Point. The west channel continues to Port Tampa and ends in a turning basin. The west channel to Gadsden Point includes the Alafia River, Port Sutton, East Bay, and Seddon Channels. The project depth varies from 45 feet in the entrance channel at the Egmont Bar Channel to 30 feet in the Alafia River. Length of the project is about 67 miles including 3.6 miles in the Alafia River. The Port of Tampa has more cargo tonnage than all other Florida ports combined.
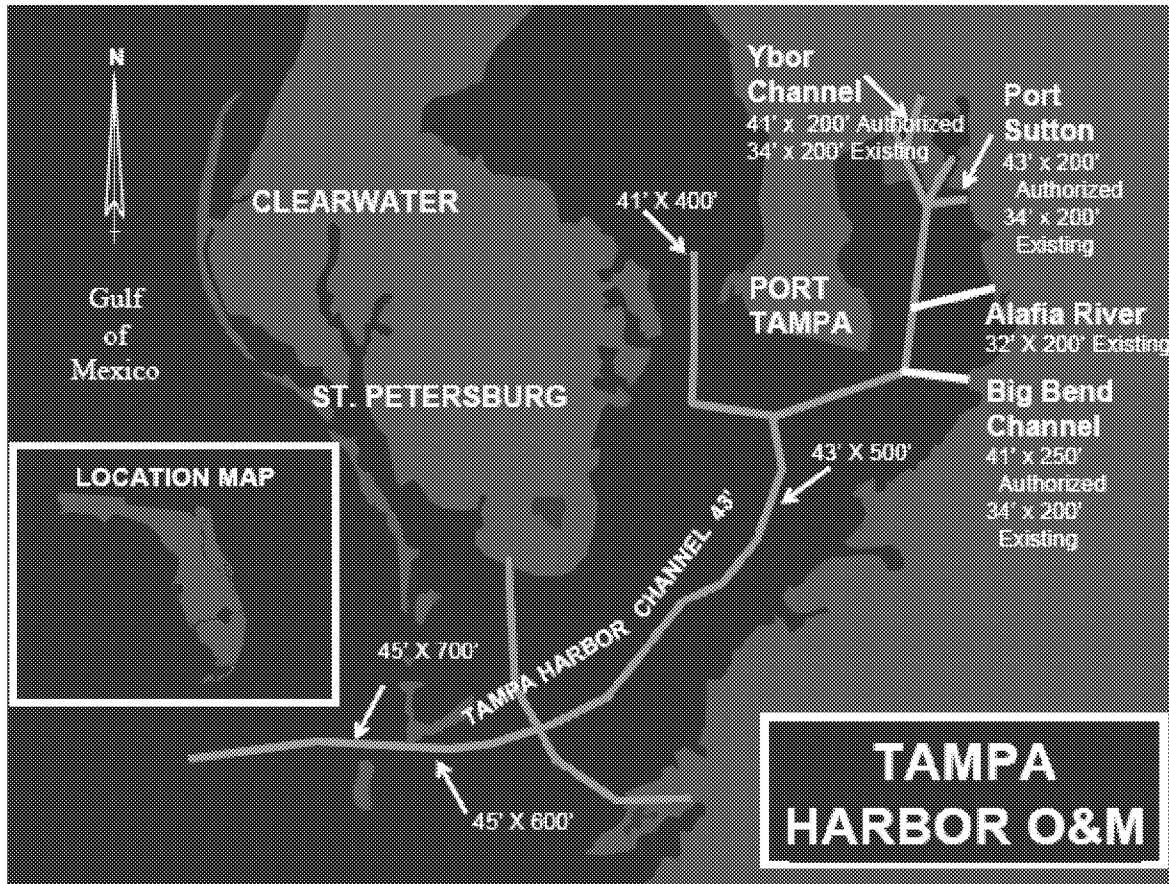
Figure D-1. Tampa Harbor Deep-Draft navigation project.

D-3. Section 1—Tampa Harbor CEPD Project Datum Evaluation Report (Jacksonville District).

US ARMY ENGINEER DISTRICT, JACKSONVILLE
Comprehensive Evaluation of Project Datums

PROJECT DATUM EVALUATION REPORT

Tampa Harbor, Florida (30 to 45-Foot Projects)
Hillsborough River (9 & 12-Foot Project)
Alafia River (30-Foot Project)

9 September 2007 (Revised 15 Oct 07)

Synopsis of Overall Project Assessment

This report assesses the adequacy and accuracy of reference datums for the Tampa Harbor Project, including all related shore protection control structures, and/or upland/offshore disposal sites associated with this project, as described in the project authorization documents. This evaluation is performed in compliance with the Commanding General's 4 December 2006 directive memorandum, subject, "Implementation of Findings from the Interagency Performance Evaluation Task Force for Evaluating Vertical Datums and Subsidence/Sea Level Rise Impacts on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects." The findings in this report are summarized below.

1. The project is NOT compliant with the standards and guidance in EC 1110-2-6065 [1].

2. The current tidal MLLW reference datum model for this project is of uncertain origin, not fully documented, and appears not to have been updated to the latest 1981-2001 sea level epoch in accordance with WRDA 92. NOAA CSDL has developed a VDatum hydrodynamic model of the MLLW gradient throughout the area. This model is not being used in USACE surveys.

3. Currently, water surface elevation corrections for dredging measurement & payment are based on extrapolated staff gage readings set from benchmarks of uncertain origin, that are not referenced to the NSRS, and/or are referenced to the superseded NGVD29 datum. Use of NOAA PORTS gage readings may be resulting in mixed tidal epochs. Recent RTK surveys have originated at NOAA tidal benchmark sites; however, survey and dredging reference are still on the superseded 1960-1978 tidal epoch. Project framework and control documents do not clearly define references or relationships between these benchmarks and NOAA tidal gages or tidal benchmarks.

4. Given VDatum coverage, no significant corrective actions will be required to hydrodynamically model the tidal regime, or model the geoid. Corrective actions will be

required to establish a fully calibrated RTK horizontal and vertical positioning network throughout the project, and update project framework documents. Recommended actions are outlined in this report.

     5. The estimated cost to effect corrective actions is $76,000.

     6. Corrective actions should be budgeted and programmed for completion in FY 2008.

     7. Estimated cost avoidance savings potentially realized in effecting corrective actions is $7,500,000.

Hydrological and Hydraulic Modeling Requirements

     1. Figure CEPD-1 shows the typical tidal correction locations currently used on a portion of this project. The reference datum is referred to MLLW as required under WRDA92. However, the tidal epoch has not been updated from 1960-1978 to 1983-2001. Water surface elevations are extrapolated from shore-based gages.
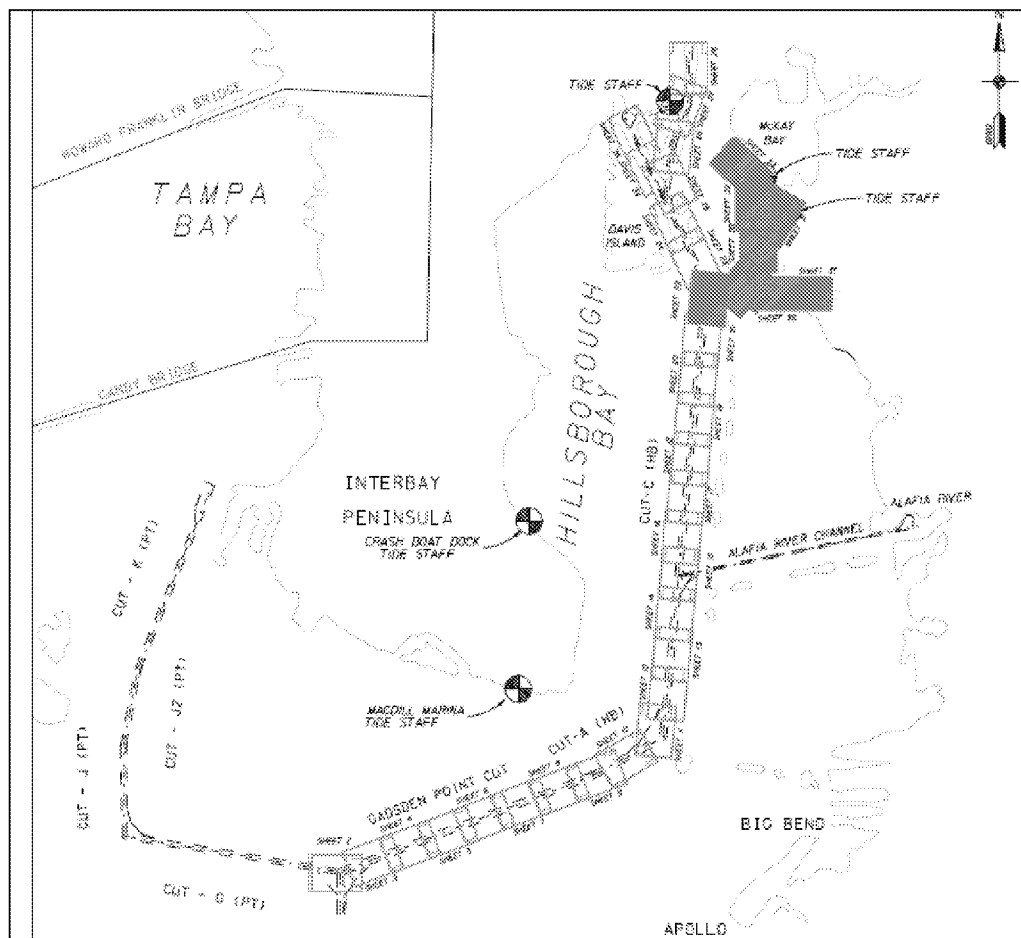

Figure CEPD-1. Typical tidal gage locations currently used in northern portion of project (07-076)—reference NGVD29 & 1960-78 epoch.

2. *Figure CEPD-2 depicts the existing NOAA CSDL VDatum model that can be used to develop MLLW-NAVD88-geoid relationships throughout this project. Note that the VDatum model is currently on the 1960-1978 tidal epoch.*
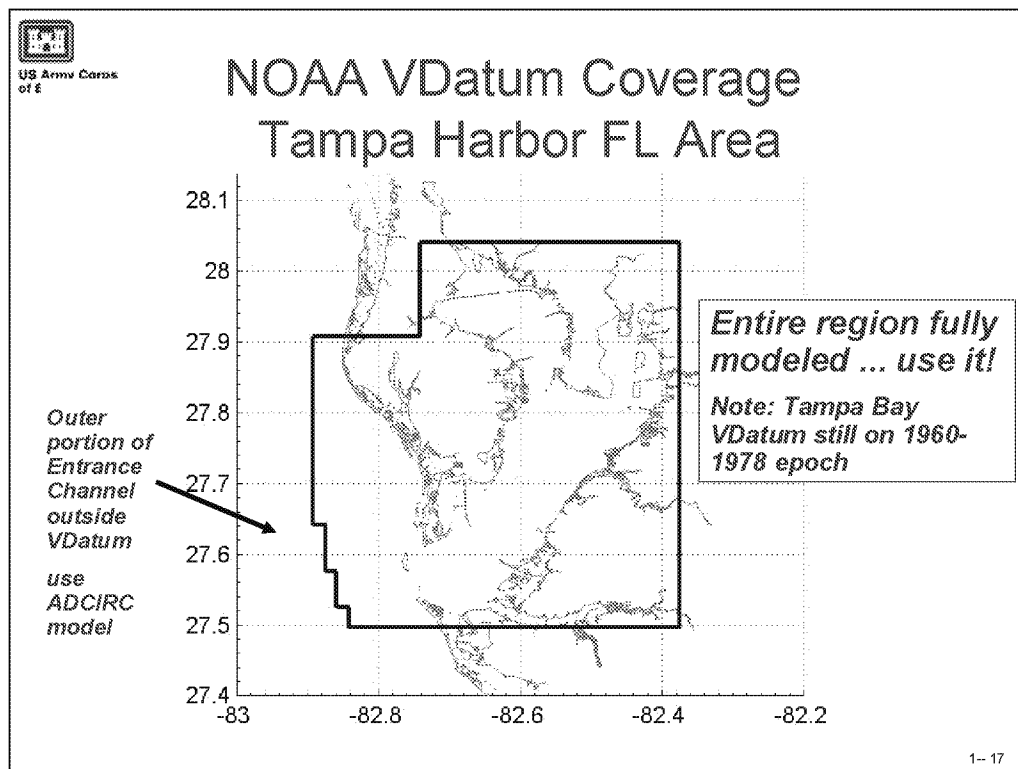

Figure CEPD-2. NOAA CSDL VDatum model coverage.

3. *Figure CEPD-3 depicts tidal bench mark gage sites maintained in the NOAA CO-OPS database. It is presumed that these gages were used to develop the VDatum model—this should be verified with NOAA CSDL. Not all these gages have been updated to the 1983-2001 epoch.*
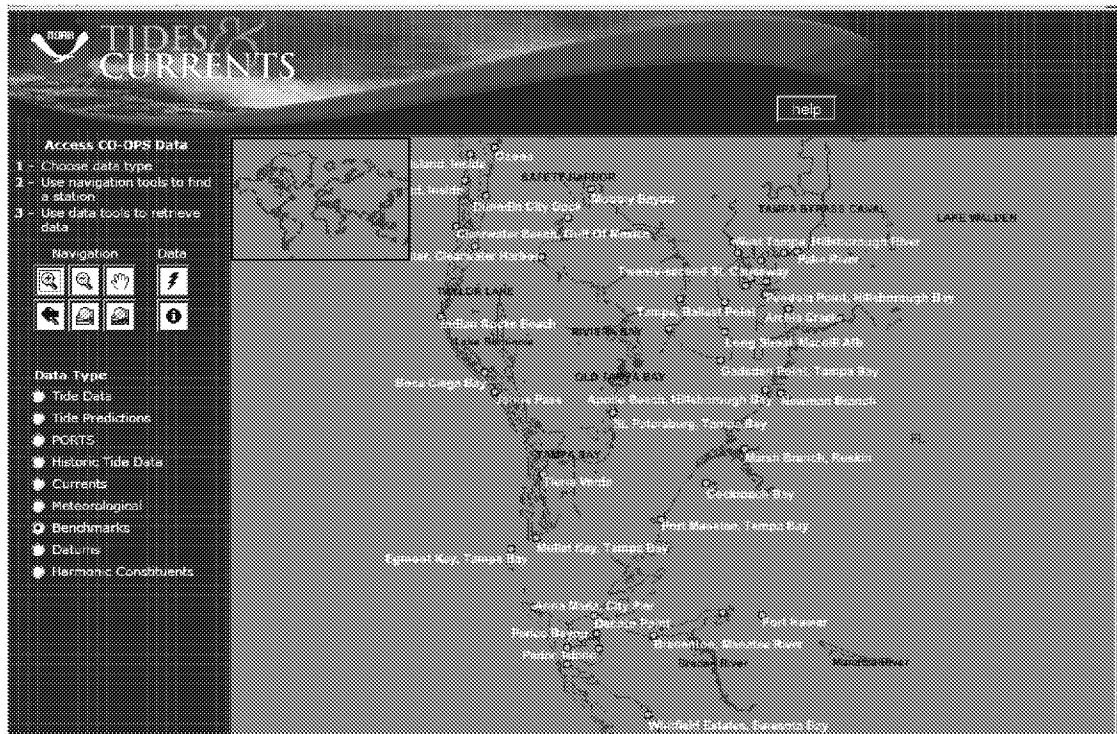
D-5

Figure CEPD-3. NOAA CO-OPS tidal bench mark sites in Tampa Harbor area.

4. *The outermost portion of Egmont Cut 1 does not have VDatum coverage. ADCIRC data may be used to estimate the tidal range gradient in this area if it is significant (i.e., gradient and/or maintenance).*

5. *No requirement for additional gages is anticipated on this project.*

6. *The 10-mile Hillsborough River shallow draft portion of the project may not warrant detailed modeling or RTK coverage. Ascertain if this area was picked up within the resolution/coverage of the VDatum model. Determine effort, if any, based on past survey/maintenance activity. (This portion of project not researched during CEPD assessment).*

7. *Actions.*

(1) *Obtain updated tidal epoch data from NOAA CO-OPS on tide stations not yet updated to 1983-2001.*

(2) *Request CSDL Update VDatum model to 1983-2001 epoch.*

(3) *From VDatum model, generate a 3D digital gridded (100 ft x 100 ft) tidal model for the entire project area depicting the relationship between MLLW (1983-2001), NAVD88, and LMSL.*

(4) RTK network calibration verification. The RTK network corrections derived from the updated model needs to be verified at NOAA tide gages in the area. Any variations need to be resolved and the model corrected accordingly.

8. Geoid Model Update Requirements

(1) Action. None—geoid heights are included in the VDatum model.

9. NOAA Tidal Gage RTK Network Calibration and NSRS Connection Requirements

(1) Sufficient NSRS high-order accuracy vertical control exists in this project area to provide NAVD88 reference for navigation measurement & payment surveys, topographic surveys of upland disposal sites, or construction surveys of coastal protection structures—see Figure CEPD-4. These NSRS control points will suffice as "Primary Project Control" PBMs in accordance with the requirements of EC 1110-2-6065 [1]. All supplemental or local project control PBMs, RTK calibration PBMs, and existing USACE PBMs deemed suitable for future use, must be directly connected to these NSRS "Primary Control" PBMs using either GPS or differential leveling methods. Reference EC 1110-2-6065 [1] for detailed survey specifications, metadata reporting, documentation requirements, and requirements for NSRS input. (Note that supplemental or local PBMs, although tied to the NSRS, do not need to be input into the NSRS; however, there may be exceptions at NOAA or Corps gage sites).
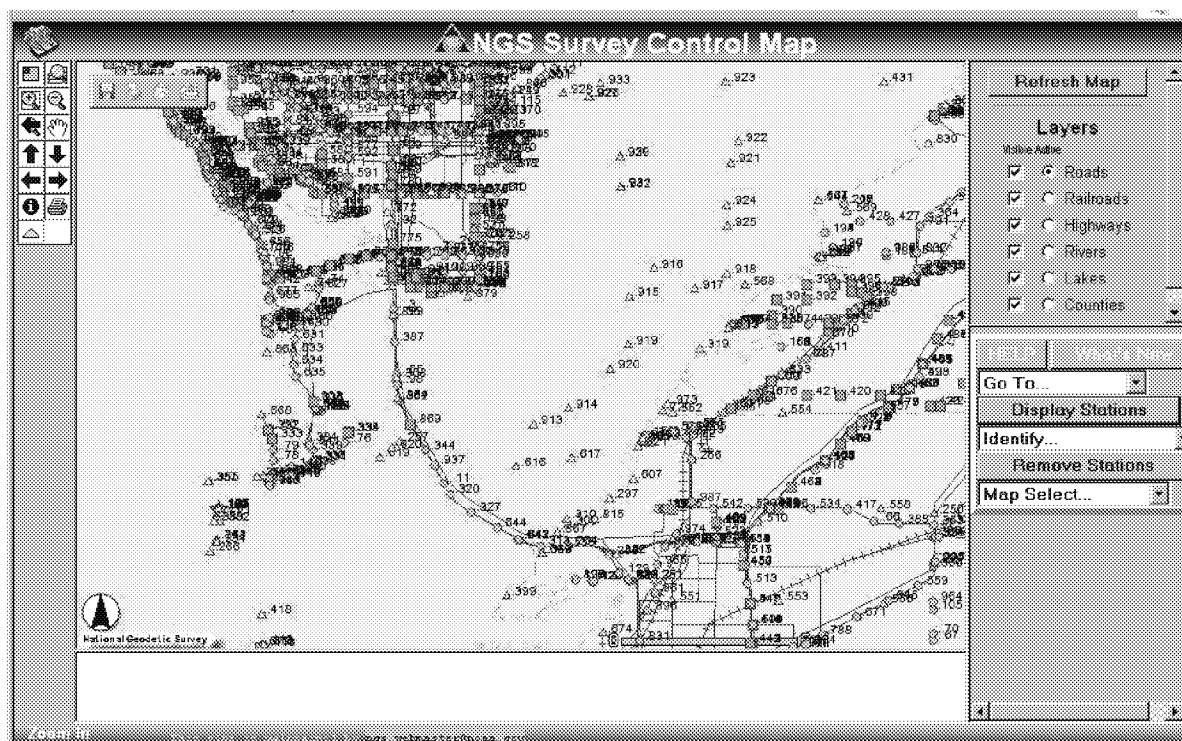

Figure CEPD-4. NGS/NSRS control data in southern region of Tampa Bay. (Squares-H+V, Circle-V)

(2) Sufficient NOAA tide gages and benchmarks (see Figures CEPD-3 and CEPD-4) exist throughout the project area to facilitate calibration of the combined tidal-geoid model used with a RTK elevation measurement system. Tide staffs should be set at NOAA gage sites on MLLW (1983-2001) to calibrate the model and for use as a QC on periodic RTK measurement & payment surveys.

10. Actions.

(1) Recover tidal benchmarks and set RTK calibration staffs at approximately eight to ten NOAA tide gage sites along the project reach; as needed to afford RTK coverage to the work sites. Follow EC 1110-2-6065 [1] bench mark recovery and documentation requirements.

(2) As required, perform full OPUS DB/PROJECT observations at tidal benchmarks at each of the above tidal gage sites, per EC 1110-2-6065 [1] specifications. No GPS observations are required if the site has NSRS NAVD88 control, or can be leveled to.

(3) Tidal benchmarks at existing NOAA CO-OPS sites are recommended as temporary RTK base stations for local dredge operations and measurement & payment surveys.

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[1] Above references to "EC 1110-2-6065" in this 2007 CEPD report refer to an interim guidance document in effect at that time. This manual replaces the interim circular]

D-4. <u>Section 2—Tampa Harbor Channel Framework Report (Jacksonville District)</u>.

Tampa Harbor Navigation Project

Master Channel Control Framework Report

21 August 2009

REFERENCES:

CESAJ-EN-DW "Procedures & Standards for Developing & Maintaining Master Channel Control Framework Documents" (7 August 2009)

CEPD PROJECT DATUM EVALUATION REPORT—Tampa Harbor, dated 9 Sep (15 Oct) 2007

2009 Project Condition Survey: (Reference File 09-083)

1. Overview

a. This report summarizes actions in developing a Master Channel Control Framework for the subject project. The scope of this report includes all channels from Egmont Cut in the Gulf into Tampa Bay and Hillsborough Bay; including the Alafia River and port areas. It does not include the 9- &12-ft Hillsborough River project.

b. The master channel control framework version was developed using a composite of the most recent PCS Survey (2009)--referenced above—and updated channel framework dgn files. The composite Master Channel Control Framework dgn file was developed using these various sources. The project has been transformed to NAD83 and the MLLW vertical reference grade updated to the current tidal epoch (1983-2001).

2. Horizontal Datum Transform

Existing project framework drawings were converted to NAD83 at some point prior to the 2007 CEPD Report—no documentation exists on this effort. It is assumed that the process outlined in EM 1110-2-1005 (and prior guidance documents) was followed and the current channel PI framework coordinates adequately define the geospatial channel alignment.

3. Vertical Datum Modeling (VDatum)

This project area is covered by a VDatum model referenced to the 1960-1978 epoch—refer to the 2007 CEPD Report. In 2009, CESAJ-EN-DG updated the current VDatum model for Tampa Bay to the latest tidal epoch (1983-2001)—a 0.20 ft average change in MLLW was assumed constant throughout the project. A Kinematic Tidal Datum (KTD) file was then created for Tampa Bay. This updated model (and KTD file) should be used until NOAA releases a new version of VDatum in FY10 which will be on the 1983-2001 epoch. A "preliminary" site calibration of the epoch-corrected VDatum model was performed at various NOAA gage sites around the perimeter of Tampa Bay. NOAA gage MLLW elevations were compared with the VDatum MLLW from RTK/RTN observations. Maximum deviations between RTK tide and gage tide observations were not more than 0.2 ft. Once the new FY10 release of VDatum is available, a final site calibration should be performed for the channel reaches, as outlined below.

4. Geoid Model

The current NGS geoid model shall be used to correct for undulations over the project. The extrapolated geoid heights shall be considered as absolute for correcting observed ellipsoid heights.

5. Construction Survey Positioning Criteria

a. Horizontal Vessel Positioning. A regional RTN network based on NSRS CORS covers most of the Tampa Bay project. Therefore, the RTN indirectly represents the PPCP(s) for this project, subject to local site calibration at a NSRS or NWLON point.

b. The following reaches are outside the RTN coverage and require an RTK base station.

Channel Reach	RTK Base	PID
Egmont Cut 1	Desoto C	AG 0489

c. Vertical Control. Vertical RTK/MLLW calibrations on the most recent surveys have been "checked" by comparisons with real-time PORTS values. When the FY10 update of VDatum is released, a complete RTK/RTN vertical calibration of "RTK Tides" against gage tide readings should be performed relative to a staff gage set to MLLW from tidal bench marks at the following NOAA gage stations.

NOAA Gage	Station ID	"RTK Tide" Calibration Check Results
Egmont Key	872 6347 or	_____
Port Manatee	872 6384	_____
St. Petersburg	872 6520	_____
Gadsden Point	872 6573	_____

Long Shoal-MacDill   *872 6604*   _____

Davis Island   *872 6657*   _____

Port Tampa   *872 6607*   _____

d. RTN/RTK observed tide levels above MLLW should ideally agree with the staff gage observations to around 0.2 ft. If these differences at a gage are consistent, then these gage-channel-zoned RTK/RTN site calibration values should be applied by all users.

6. Master Channel Control Framework Drawing Notes

The following notes shall be placed on the master channel control framework drawing.

HORIZONTAL REFERENCE SYSTEM:
THE HORIZONTAL REFERENCE DATUM FOR THIS PROJECT IS NAD83, BASED ON THE CURRENT VERSION OF THE NOAA NATIONAL SPATIAL REFERENCE SYSTEM (NSRS). GRID COORDINATES ARE SHOWN IN THE FLORIDA STATE PLANE COORDINATE SYSTEM (SPCS)—WEST ZONE (0902). CHANNEL STATIONING AND OFFSET COORDINATES ARE RELATIVE TO THE INDICATED CHANNEL BASELINE FOR EACH CHANNEL REACH. CHANNEL ALIGNMENTS ARE GRID BEARINGS REFERENCED TO THE SPCS GRID. UNLESS OTHERWISE INDICATED, CHANNEL WIDTHS AND LIMITS CONFORM TO THE AUTHORIZED PROJECT DIMENSIONS.

VERTICAL REFERENCE SYSTEM:
THE TIDAL REFERENCE GRADE FOR THIS PROJECT IS MEAN LOWER LOW WATER (MLLW), BASED ON THE NOAA 1983-2001 NATIONAL TIDAL DATUM EPOCH. THE NAVD88-MLLW RELATIONSHIP ON THIS PROJECT HAS BEEN HYDRODYNAMICALLY MODELED USING NOAA VDATUM--REFERENCE "TAMPA HARBOR FRAMEWORK REPORT." THE ESTIMATED LOCAL (RELATIVE) ACCURACY OF THIS TIDAL MODEL IS ± 0.1 FT.

CONSTRUCTION SURVEY POSITIONING CRITERIA:
HORIZONTAL POSITIONING AND WATER SURFACE ELEVATION MEASUREMENTS (INCLUDING CALIBRATIONS) SHALL BE PERFORMED UTILIZING REAL-TIME KINEMATIC (RTK) OR RTN GPS OBSERVATIONS FROM (OR CALIBRATED TO) THE FOLLOWING PRIMARY REFERENCE PBMS. PBM COORDINATES AND TIDAL PBM MLLW ELEVATION DATA SHALL BE OBTAINED FROM THE CURRENT NOAA NSRS AND NWLON DATABASES.

MLLW CALIBRATION GAGES (NOTE THAT SOME GAGES MAY HAVE SITE CALIBRATION ADJUSTMENTS):

Channel Reach	NOAA Gage	Station ID
Egmont Cuts	Egmont Key	872 6347 or
Mullet Key Cut	Mullet Key	872 6364
Cut A, Cut B, and Cut C	Port Manatee	872 6384
Cut D, Cut E, and Cut F	St. Petersburg	872 6520
Gadsden Point Cut to PI Cut A & C (HB) and Cut G (PT)	Gadsden Point	872 6573
Cut C (HB) and Alafia River Channel	Long Shoal-MacDill	872 6604
Davis Island, Seddon Island	Ballast Point	872 6639 or
Port Sutton, & McKay Bay Channels	Hooker Point	872 6668 or
	Davis Island	872 6657
Cut J (PT) & Cut K (PT)	Port Tampa	872 6607

RTK BASE STATIONS OUTSIDE RTN COVERAGE:

Channel Reach	RTK Base	PID
Egmont Cut 1	Desoto C	AG 0489

THE SPATIALLY MODELED NAVD88-MLLW RELATIONSHIPS FOR EACH CHANNEL REACH HAVE BEEN INCORPORATED INTO A KINEMATIC TIDAL DATUM MODEL FOR TAMPA HARBOR. THIS KTD FILE INCORPORATES A 0.20 FT EPOCH CORRECTION INTO THE NOAA PUBLISHED VDATUM MODEL ON THE 1960-1978 EPOCH. THIS CESAJ-OD-H KTD MODEL SHALL BE USED TO CORRECT MEASURED RTK/RTN ELLIPSOID HEIGHTS FOR NAVD88-MLLW DATUM AND GEOID HEIGHT VARIATIONS; FOR SURVEY OPERATIONS PERFORMED ON THIS PROJECT. [AN UPDATED NOAA VDATUM MODEL IS EXPECTED IN 2010].

REFERENCES:

"TAMPA HARBOR FRAMEWORK REPORT," VERSION DATED 21 AUG 09. MASTER CHANNEL FRAMEWORK FILE [TampaHbrV-SPmccf.dgn] VERSION ___ ___ 09 AUTHORIZATION DATA: (REFER TO TAMPA HARBOR FRAMEWORK REPORT) LOCATION OF REFERENCES AND KTD FILE: PROJECTWISE\CONTROL DATA\NAVIGATION PROJECTS\TAMPA HARBOR

A sample portion of the KTD file generated for Tampa Bay is shown below. The values in the file represent the differences between NAVD88 and MLLW (1983-2001), at the grid points in the file. NAVD88-MLLW differences throughout the entire Tampa Bay region vary some 0.3 ft. This KTD file would be provided to all users, along with any site calibration gage constants.

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41 41 335000 1175000 535000 1375000    [SPCS Grid limits]
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 +1.6 +1.6 +1.59 +1.56 +1.56 +1.57 +1.57 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 +1.75
+1.75 +1.75 0 0 0 0 0 0 0 0 0 0 0
 +1.6 +1.6 +1.57 +1.54 +1.53 +1.54 +1.55 +1.55 +1.55 +1.54 +1.55 +1.56 0 0 0 0 0 0 0 0
0 0 0 0 0 +1.73 +1.74 +1.75 +1.75 +1.75 +1.75 0 0 0 0 0 0 0 0 0
 +1.6 +1.58 +1.55 +1.53 +1.52 +1.53 +1.53 +1.54 +1.55 +1.55 +1.53 +1.54 0 0 0 0 0 0 0
0 0 0 0 0 +1.7 +1.71 +1.72 +1.74 +1.75 +1.75 +1.75 +1.75 0 0 0 0 0 0 0 0
 +1.59 +1.57 +1.55 +1.54 +1.53 +1.53 +1.54 +1.54 +1.55 +1.54 +1.53 +1.53 0 0 0 0 0 0 0
0 0 0 0 0 +1.69 +1.7 +1.71 +1.73 +1.74 +1.75 +1.75 +1.75 +1.75 0 0 0 0 0 0 0 0
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APPENDIX E

Tidal Modeling Procedures for Coastal Navigation Projects

E-1. Purpose. This appendix provides supplemental guidance to Chapter 4. It contains technical guidance and examples of interpolated tidal modeling procedures for USACE coastal navigation projects. It is primarily applicable in those regions where NOAA VDatum model coverage is not complete (as of 2009) or does not exist, such as in intracoastal waterways and sounds.

E-2. Requirements for Accurately Modeled Tidal Reference Datums. Tidal reference datums vary both spatially and temporally. Thus, the water surface elevation at a shore-based gage is adequate only for that specific location and time. The height of the water level will be significantly different between two points around an inlet, due to varying times and weather conditions—see Figure E-1. Likewise the MLLW datum will vary with the tidal range variations, which are modified by the topography of an inlet or coastal region. The MLLW datum elevation at a reference gage should not be extrapolated to another location without some modeled correction. It is also subject to long-term variation due to sea level change, subsidence, or other factors. This requires periodic updating of tidal datums based on NOAA's latest NTDE, which is currently 1983-2001 for most areas.

    a. Tidal datum models. Lack of accurately modeled tidal datums can have significant impacts on navigation project construction and maintenance costs. Navigation projects that are not adequately referenced to an established tide gage and modeled MLLW datum plane, or have not been updated for sea level change, can result in overdredging or underdredging, along with increased construction disputes and claims. The primary factors that need to be considered in evaluating the adequacy of depth grade determination on a navigation project include the following:

    (1) Tidal range variations over the project reach. Geospatial variation of MLLW dredging datum relative to the orthometric and ellipsoid datums used to reference acoustic depth measurements.

    (2) Tidal phase variations over the project reach. Real-time survey techniques used to measure the elevation of the water surface at the construction site that corrects tidal phase variations observed at a reference tide gage.

    (3) Tidal epoch adjustments for sea level or land subsidence changes. Involves monitoring NOAA tide gage records for changes to tidal epochs, tidal PBM adjustments, etc.

    (4) Quality of reference tidal datum determinations. Awareness of the uncertainties in tidal gage datums and any models derived therefrom—i.e., VDatum.

    b. Tide gage extrapolations. The long-established practice for dredging and related payment surveys of navigation projects involves extrapolation of a water (tide) level gage to the

offshore construction site—i.e., extrapolating the tide level at the gage to the offshore point. This assumes both the water surface level and reference datum range are constant over the extrapolated distance—i.e., assumes no tidal phase or range variations exist. This distance may range from a few hundred feet to over 10 miles. These assumptions of linearity in water surface levels and datum degrade with distance from the reference gage. At low tidal ranges, longer extrapolations may be possible. At higher ranges (> 2 ft), extrapolations greater than a half-mile to 1 mile may be invalid and inaccurate. In addition, local weather conditions may further degrade the distance that a tide reading can be reliably extrapolated from a gage. Sea surface setup due to strong winds can significantly alter the surface model. Approximate methods for estimating tide phase differences ("tidal zoning") are used in some Districts, with mixed accuracy results as these methods do not account for local weather conditions.
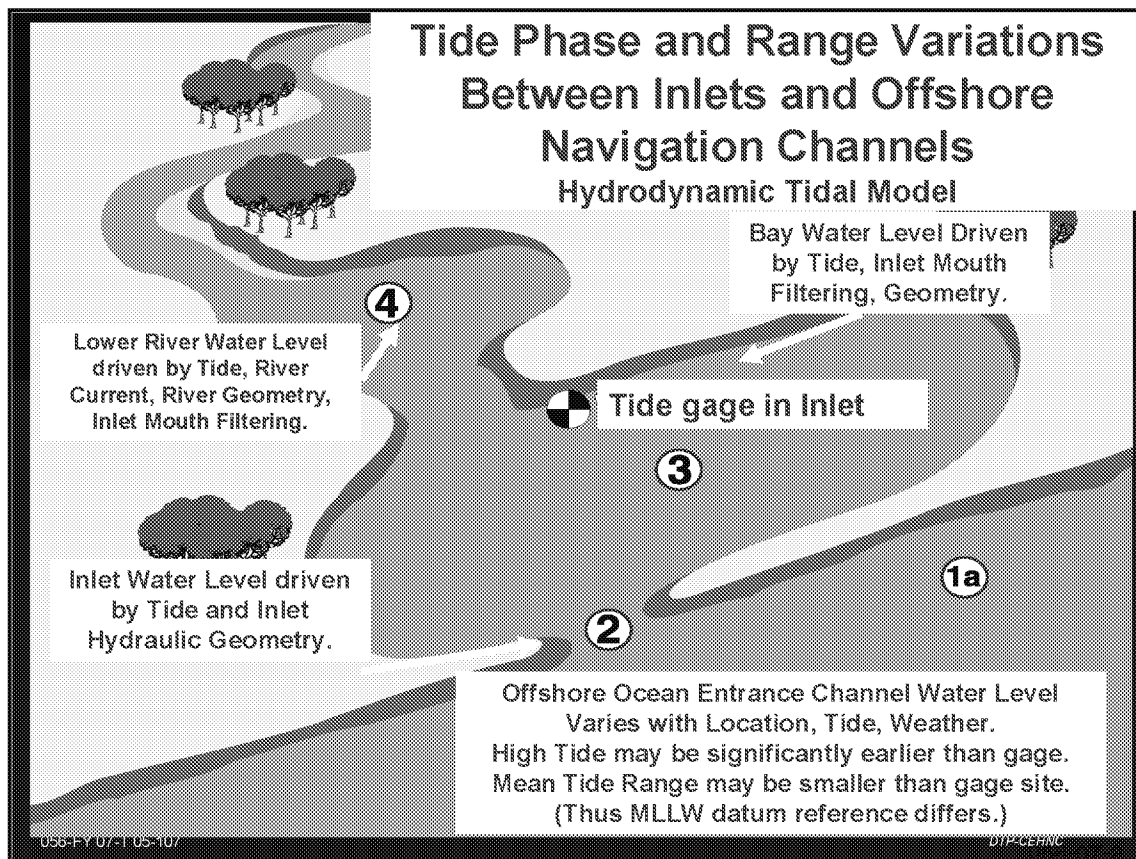

Figure E-1. Tide phase and range variations at an inlet.

c. Hydrodynamic conditions at tidal inlets. The overall effect of adverse conditions encountered at typical USACE coastal inlets is best summarized in the following excerpt from Part II of EM 1110-2-1100 (*Coastal Engineering Manual*).

"*Hydrodynamic conditions at tidal inlets can vary from a relatively simple ebb-and-flood tidal system to a very complex one in which tide, wind stress, freshwater influx, and wind*

waves (4- to 25-sec periods) have significant forcing effects on the system ... Flow enters the bay (or lagoon) through a constricted entrance, which is a relatively deep notch (usually 4 to 20 m at the deepest point). Entrance occurs after flow has traversed over a shallow shoal region where the flow pattern may be very complex due to the combined interaction of the tidal-generated current, currents due to waves breaking on the shallow shoal areas, wind-stress currents, and currents approaching the inlet due to wave breaking on adjacent beaches .... Particularly during stormy conditions with strong winds, flow patterns may be highly complex. Also, the complicated two-dimensional flow pattern is further confounded because currents transverse to the coast tend to influence the propagation of waves, in some cases blocking them and causing them to break ... Final complications are structures such as jetties, which cause wave diffraction patterns and reflections. In inlets with large open bays and small tidal amplitudes, flows can be dominated by wind stress. In such cases, ebb conditions can last for days when winds pile up water near the bay side of the inlet, or long floods can occur when winds force bay water away from the inlet. Most inlet bays, however, are small and some are highly vegetated, so wind stress is not a dominant feature, except under storm conditions ... Although many bays do not receive much fresh water relative to the volume of tidal flow, substantial freshwater input due to river flow can sometimes create vertically stratified flows through a tidal inlet. Typically, however, well-mixed conditions exist for most inlets."

E-3. <u>Modeling the MLLW Dredging Datum on USACE Navigation Projects</u>. Most often, linear or surface interpolations between gages will be used. On projects with larger tide ranges where the uncertainty of a linear model between gages increases beyond the allowable tolerance, a more sophisticated hydrodynamic model may be required to best define the MLLW datum. This presumes adequate gage records exist from which to calibrate the tidal model in an area. On some projects, a single gage may be adequate. Others may require additional gages to define and verify the model. If these additional gages do not exist, then a gaging program will have to be programmed. In addition, topographic and bathymetric models of the project may have to be generated if they do not exist. A firm connection to the orthometric datum (NAVD88) may also be required. Thus, a number of project-specific technical factors will govern the overall effort required to model the MLLW datum plane of a project. This will also include the experience of those assessing the tidal model relative to the required relative accuracy of the tidal model.

a. Tidal error budget. One must not lose sight of the overall error budget in evaluating the effort required to model the MLLW datum on a project. Relative to removing large phase and wind setup errors with RTK measurements, these MLLW datum modeling errors are often insignificant. Thus, before embarking on any extensive and costly gaging program, the significance or sensitivity of these added gage observations on the overall tidal model must be substantiated. Likewise, the difference between a simple linear interpolation and a hydrodynamically modeled interpolation must be evaluated for significance relative to the intended tolerance. In addition, there is no point in performing elaborate MLLW datum tidal modeling unless RTK surface elevation measurements are mandated for the completed project. Having a MLLW tidal model accurate to ±0.1 ft with a ±1 ft phase error due to extrapolated gage readings five miles offshore would obviously be an inconsistent use of resources.

b. Tidal model accuracy. An evaluation of the errors in a tidal model is necessary to evaluate uncertainties between tide stations. The primary factors that need to be considered in modeling a reference tidal datum around a coastal protection structure include the following:

(1) Tidal phase variations over the project reach.

(2) Tidal range variations over the project reach.

(3) Tidal epoch adjustments for sea level or land subsidence changes.

(4) Quality of reference tidal gage datum determinations

(5) Seasonal variations in LMSL (i.e., biased sea level rise during hurricane season).

(6) Need for short-term (i.e., 5-year) tidal epochs in high subsidence or uplift areas.

c. Modeling tide range or MLLW variations over a navigation project. Variations in tidal range (i.e., undulations in MLLW datum relative to MSL or to the local geodetic/orthometric datum) within a project must be accounted for. This requires developing some model of the tidal hydrodynamic characteristics throughout the project. Figure E-2 illustrates this MLLW variation over a Jacksonville District deep-draft coastal inlet project (St Johns River—Ocean to Jacksonville, FL). The MLLW datum relative to MSL varies from the ocean through the entrance jetties and up river some 20 miles to the termination of the deep-draft project past the gage at Longbranch, and further upstream in the shallow-draft project to Palatka, FL. The MSL reference plane also varies relative to NAVD88, generally rising upstream. Figure E-2 also depicts that NGVD29 and NAVD88 are not parallel datums. The MSL-MLLW datum variation may also be impacted by fresh water flow into the tidal area.

(1) Modeling the MLLW datum through a navigation project requires an adequate density of tide gages from which the model can be calibrated, and intermediate datum variations between the gages can be modeled. In Figure E-2, the roughly 5.6 ft tide range at the ocean narrows down to 1.6 ft over a 25-mile navigation project. Although the gages shown in Figure E-2 are spaced at about every 5 to 10 miles, they should be of sufficient density to calibrate a hydrodynamic tidal model for this project. The interpolations between the gages shown on Figure E-2 represent only a crude tidal model of the MLLW reference plane—a full hydrodynamic tidal model such as VDatum would be represented by a smooth curve. In many cases with small tidal range variations, or with a dense gage network, a linearly interpolated model may prove adequate. That may be the case for portions of the above project where the variation between gages is not large.

(2) Figure E-3 illustrates the tidal range variation over seven miles of a Norfolk District shallow draft project on the Atlantic east coast in Virginia. There would appear to be a sufficient density of gage data to model the MLLW datum plane for this project—updating the older MLW datum. The NGVD29 orthometric datum reference on this project needs to be updated to NAVD88 along with a NAD83/GRS80 ellipsoid reference. Note that the relationship to the legacy datum (Corps of Engineers Low Water— C.E.L.W.) is shown on the figure.
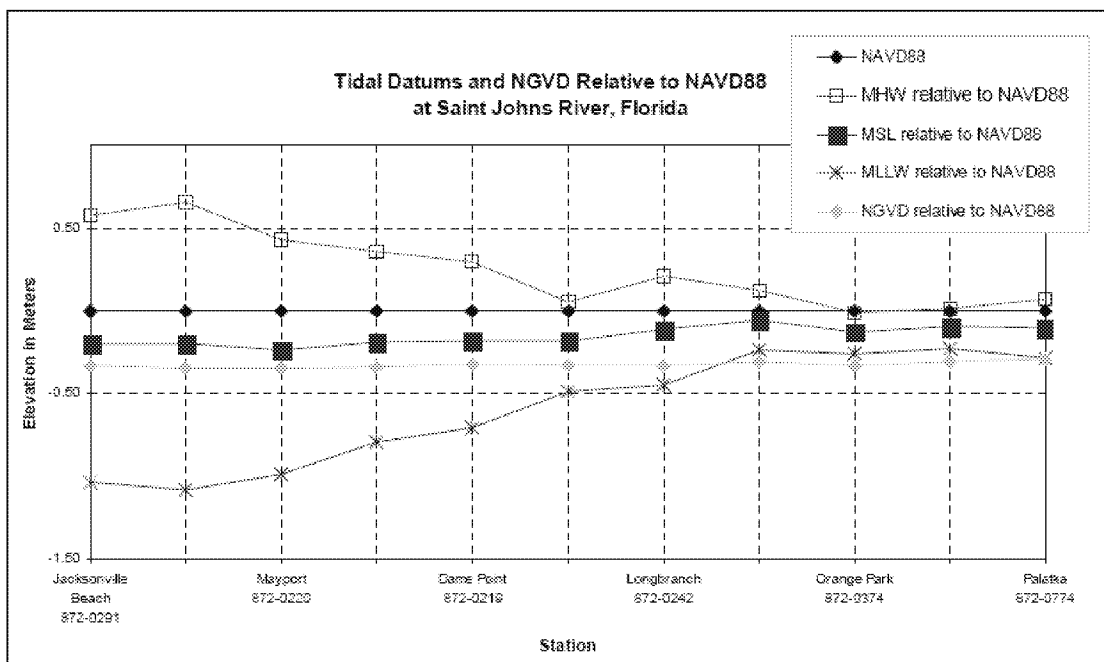
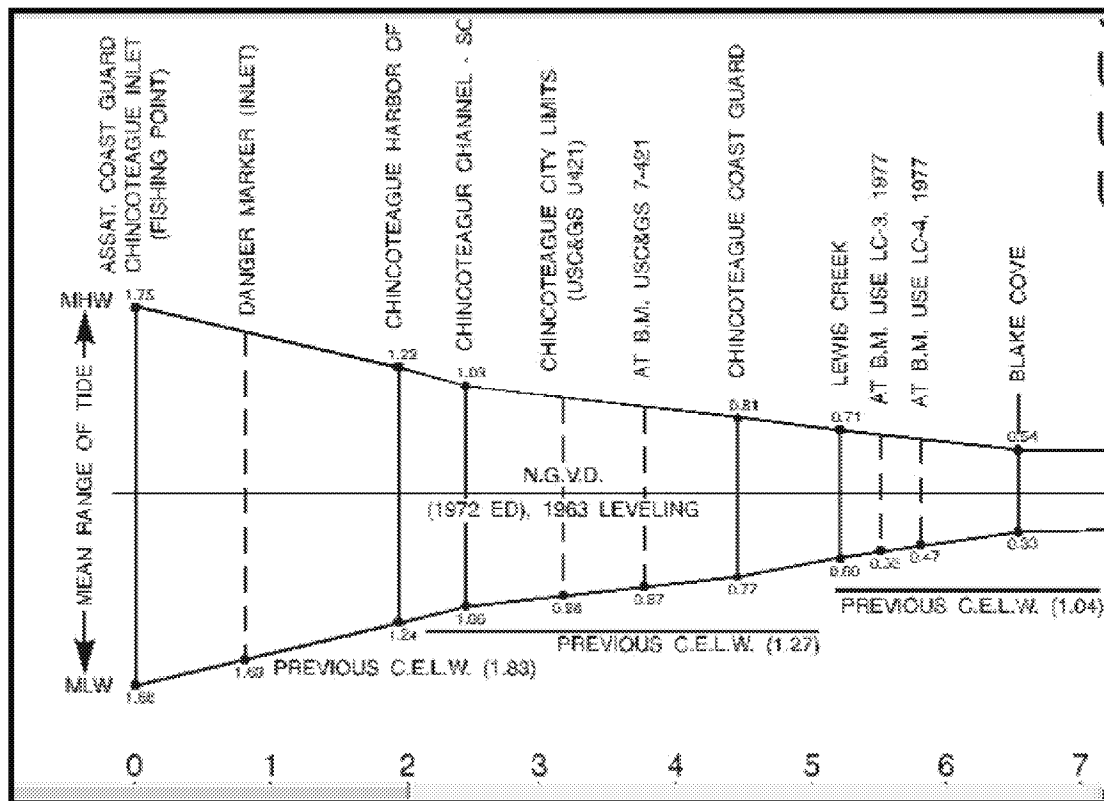Figure E-2. Tidal range variation at a coastal inlet.

Figure E-3. Tidal range variation at Chincoteague Inlet, VA.

E-4. Procedures for Estimating Navigation Project MLLW Datum Models using Spatial Interpolation Techniques.

a. Tidal model updating actions. A number of options exist to update a tidal model for coastal navigation projects that require upgrading to the current NOAA tidal reference. Updating the tidal model requires the following basic actions: (1) ensure tidal datum reference planes (MLLW) are defined relative to published NOAA gages and tidal bench marks, (2) ensure the latest NTDE adjusted by NOAA is used, (3) model the MLLW reference plane relative to NAVD88 throughout the length of the project, (4) publish and disseminate the NAVD88-MLLW model for users, (5) optionally develop the NAVD88-MLLW datum relationship at tidal bench marks if these marks are used as RTK base stations, and (6) submit any USACE hydrodynamic tidal modeling data to NOAA for their use in expanding the nationwide VDatum.

b. Tidal gaging options. Items (1) and (2) above are easily achieved as long as an existing or historical gage exists at the navigation project. This will likely be the case for the majority of the Corps' deep-draft navigation projects. If not, then a standard gaging program will have to be developed in order to establish a tidal datum at a project—see "*Computational Techniques for Tidal Datums Handbook*" (NOAA 2003). Any such effort must be coordinated with NOAA in order to ensure the project becomes included in NOAA's NWLON inventory. Time and cost estimates for performing the gaging can be obtained from NOAA. Project modeling—Items (3) through (6) above—will require close coordination with District H&H elements, ERDC/CHL, and/or NOAA. In small tide ranges either between gages or in the overall area, linear interpolation of the MLLW model will often be sufficiently accurate and economically developed. These models may already have been developed for some projects, and may currently need only to be adjusted for tidal epoch updates and geoid models.

c. Example of model vs. interpolation decisions--Miami Harbor (Jacksonville District). Figure E-4 depicts a navigation project (Miami Harbor) where a simple straight-line interpolation of the tidal datum might be warranted in lieu of performing a full hydrodynamic model study. Initial estimates of changes in time and range of tide for any survey area can be obtained from a review of the NOAA tide prediction "Table 2" information found on the NOAA CO-OPS web site. (This web site also provides links to NGS bench mark datasheets). The NOAA tide table values should be used with caution as the data summaries are from observations of varying lengths and various time periods and may be out of date and no longer reflective of current conditions. The tide tables list mean ranges of tide (MHW – MLW), Spring Ranges of Tide (Range of tide at New and Full moons), and the elevation of Mean Tide Level (MTL) above Chart Datum (MLLW). NOAA tide prediction data for the Miami Harbor area is shown below (in feet).

	Lat	Long	Mn Rge	Spg Rge	MTL
Miami Harbor Entrance	25° 46.1'	80° 07.9'	2.46	2.93	1.39
Government Cut,					
Miami Harbor					
Entrance	25° 45.8'	80° 07.8'	2.32	2.83	1.32
Biscayne Bay					
San Marino Island	25° 47.6'	80° 09.8'	2.14	2.57	1.21
Miami, Marina	25° 46.7'	80° 11.1'	2.18	2.59	1.22
Dodge Island,					
Fishermans Channel	25° 46.2'	80° 10.1'	2.10	2.52	1.19
Dinner Key Marina	25° 43.6'	80° 14.2'	1.94	2.33	1.10

This navigation project has an adequate density of NOAA tide data and has a relatively small tidal range—around 2.5 ft at the ocean entrance. The mean range of tide decreases by 0.16 ft between the Miami Beach Government Cut and inside near the Port of Miami turning basin. Similarly, the 0.14 ft range decrease is small between outside on Miami Beach and Miami Beach Government Cut. The regionally modeled tidal range at a point 3 miles offshore in open ocean could be compared with the range at the Miami Beach pier to see if there is a significant difference. The slope of MLLW can be estimated by looking at the changes in the elevation of MTL relative to MLLW. On the outside, the MTL-MLLW difference is approximately 1.4 ft and decreases to approximate 1.2 ft. inside at the Miami Marina (see Figure E-4). Given the small tide range, and the relatively small tidal range variations between outside and inside channels, the complexity of the variations is not sufficient to warrant a development of a hydrodynamic model. Thus, a straight-line interpolation of the model between observation locations would be acceptable. A regional ocean tidal model (e.g., the ADCIRC 2001 East Coast Model) would be considered in assigning a range value to the model for the outer offshore end of the entrance channel.
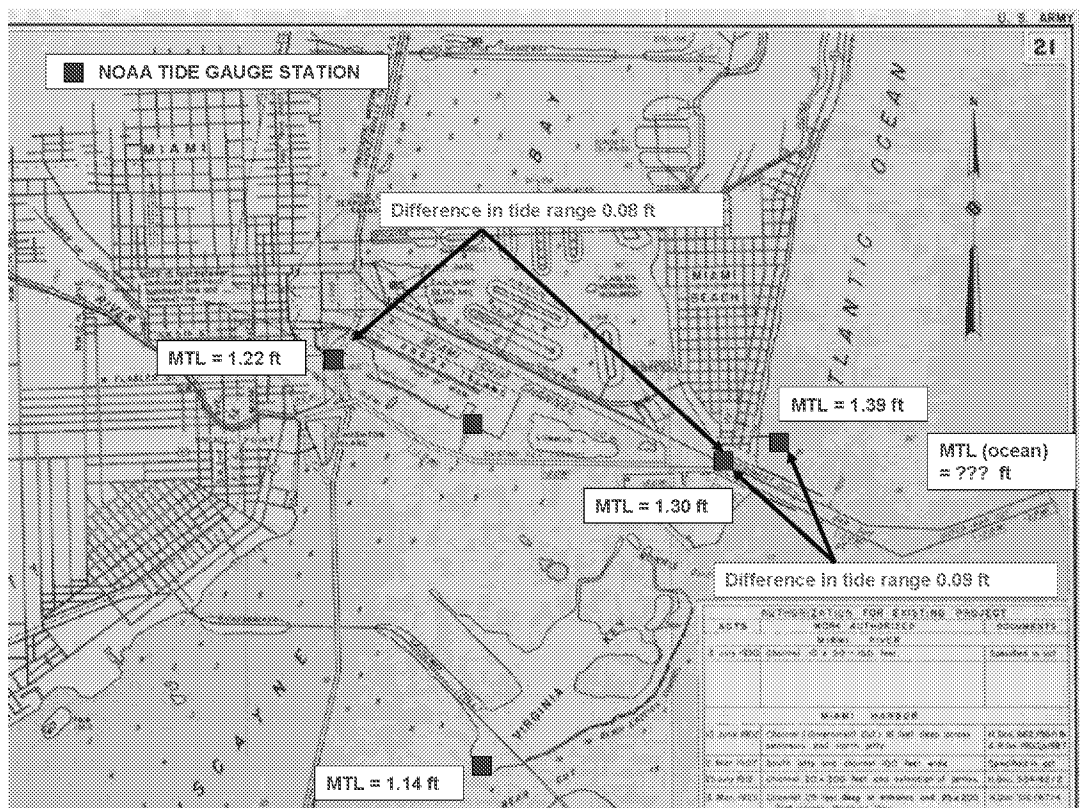
Figure E-4. Tidal Model calibrations at Miami Harbor.

d. Example: Yaquina River (Portland District). A similar analysis can be made for a West Coast project with a larger tide range—Yaquina River, OR (Portland District). The authorized depth varies from 40-ft at the bar, to 18 ft at Yaquina, then 10-ft to Toledo. The estimated mean range of tide and the MTL-MLLW elevation differences from the NOAA tide tables are shown below (in feet):

Yaquina Bay and River	Lat	Long	Mn Rge	Spg Rge	MTL
Bar at entrance	44° 37'	124° 05'	5.9	7.9	4.2
Newport	44° 38'	124° 03'	6.0	8.0	4.3
Southbeach	44° 37.5'	124° 02.6'	6.37	8.34	4.51
Yaquina	44° 36'	124° 01'	6.2	8.2	4.4
Winant	44° 35'	124° 00'	6.3	8.2	4.3
Toledo	44° 37'	123° 56'	6.3	8.1	4.2

However, a check of the latest NOAA tide station published bench mark information shows that the tide table values are out-of-date and should not be used. In general, if the latitude/longitude files have values only to the nearest degree, as opposed to a tenth of a degree, then the data are from pre-1960 observations. Using the latest information collected in the 1980's by CO-OPS, the table becomes (in feet):

Yaquina Bay and River	Lat	Lon	Mn Rge	MTL
Bar at entrance	44 37	124 05	5.9	4.2
Newport	44 36.6	124 03.3	6.21	4.49
Southbeach	44 37.5	124 02.6	6.26	4.51
Weiser Point	44 35.6	124 00.5	6.46	4.57
Toledo	44 37.0	123 56.2	6.87	4.71

Thus the older results show much less variability in the tide range than the updated, more recent data. The table and Figure E-5 shows that the range of tide increases by almost 1.0 ft. from outside to upriver at Toledo, and there is a 0.50 ft. slope in MLLW relative to MTL. This may be an area where a hydrodynamic model may prove useful to account for the non-linear changes in the tide going upriver.
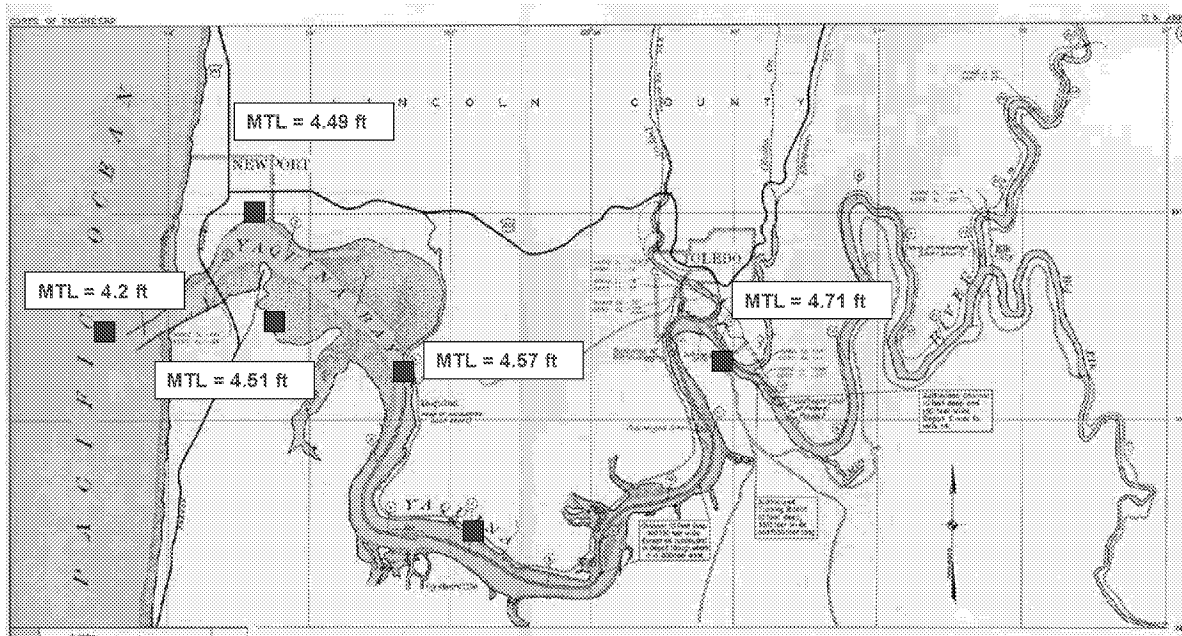
Figure E-5. Tidal Model calibrations at Yaquina River, OR.

e. Example: Portsmouth, NH (New England District). The following New England District project (Portsmouth, NH) is typical of a large tidal range variance—approximately 8 ft. MTL variations at various points are shown in Figure E-6. Predicted tide ranges are shown below.

Portsmouth Harbour	Lat	Long	Mn Rge	Spg Rge	MTL
Jaffrey Point	43° 03.4'	70° 43.9'	8.7	10.0	4.7
Gerrish Island	43° 04.0'	70° 41.7'	8.7	10.0	4.7
Fort Point	43° 04.3'	70° 42.7'	8.6	9.9	4.6
Kittery Point	43° 04.9'	70° 42.2'	8.7	10.0	4.7
Seavey Island	43° 05'	70° 45'	8.1	9.4	4.4
Portsmouth	43° 04.7'	70° 45.1'	7.8	9.0	4.2
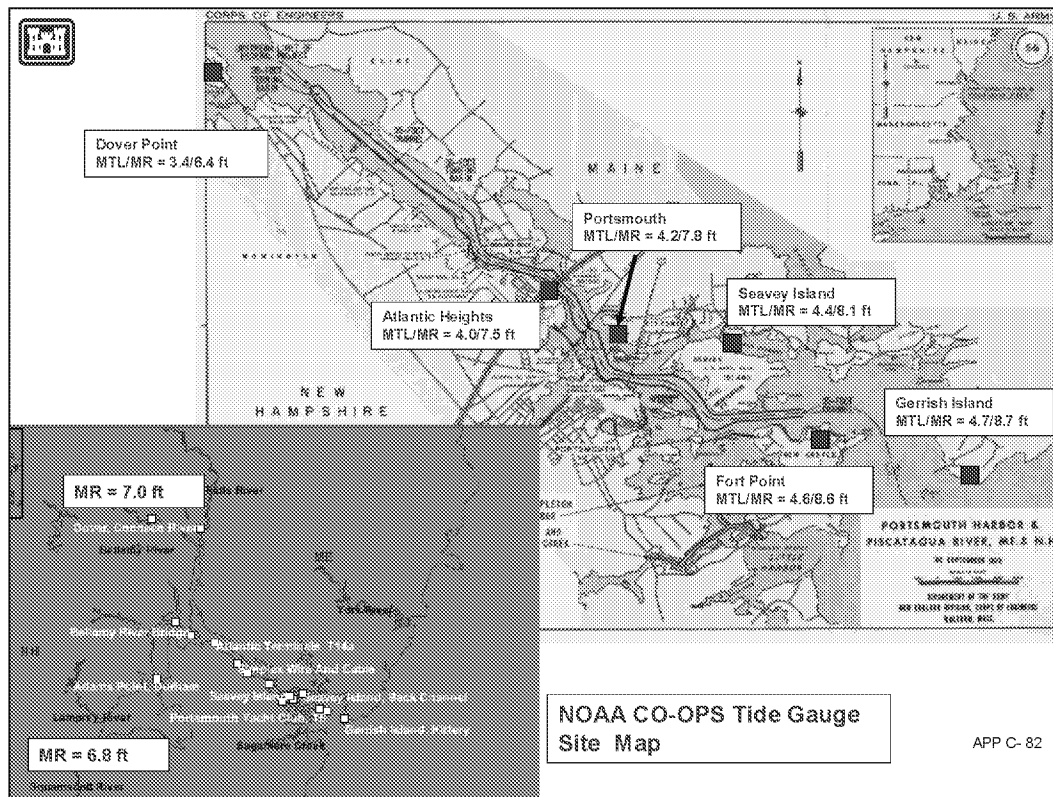
Figure E-6. Tidal Model calibrations at Portsmouth, NH.

E-5. Tidal Zoning Models. Discrete tidal zones are constructed based on knowledge of the tide at shore-based historical stations and estimated positions of co-tidal lines for range and time of tide. For most NOAA applications the resolution of the zoning has been to construct a zone polygon for every 0.2-foot change in range and every 0.3-hour change in time of tide. For many tidally complex areas (such as around Key West for instance) tide zones with higher resolution are used. Tidal zoning errors are considered random errors although they have a certain periodic nature and not a normal statistical distribution. Zoning errors also are characterized by two components: a time correction and a range ratio correction to observations from a nearby tide station. Maximum zoning errors for each project are estimated by simultaneously comparing tide curves constructed from time and range corrections to historical tide station observations. Statistics of the residuals are then analyzed to estimate the error in the zoning for the entire project.
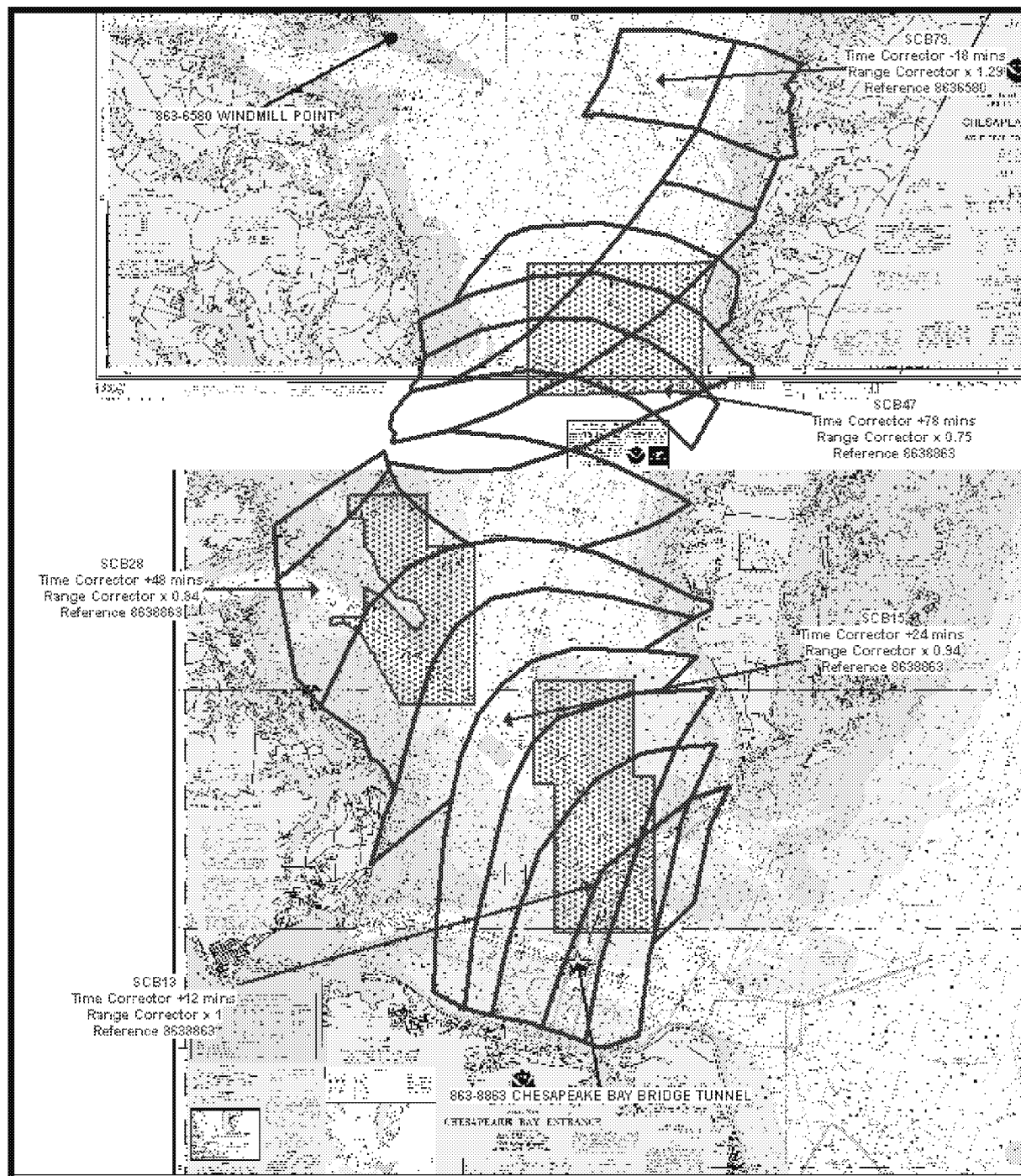
Figure E-7. The discrete tidal zones constructed from the co-tidal lines and the survey areas in lower Chesapeake Bay.

a. Tidal zoning errors. There are inherent errors in the application of discrete tidal zoning: 1) discontinuities at the edge of the zones; 2) resolution in areas of complex tidal characteristics, where the location and number of zones is not adequate to describe the changes in the tide over the survey area; 3) where large time corrections and large range ratios are required; and 4) the

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fact that placement of the zones becomes subjective when the co-tidal lines are based upon inconsistent or inadequate source data. Figure E-7 illustrates an application for tidal zoning in Chesapeake Bay—in particular for areas in the middle of the bay where no RTK or RTN coverage is available. Where RTK/RTN coverage is available only the co-range model would have application.

b. Discussion of applications. The major contributors to the tides error budget are the datum error, which contributes as a systematic bias, and the tidal zoning error, which contributes as a random error. In practice the datum error is reduced with longer data series. Errors can be very significant if less than 30-days of data are observed. Substantial reductions in error from those of a 30-day series are not realized until one-year of data are collected. For tidal modeling purposes, NOAA gage datums, (or acceptable datums from another agency's long-term gages) will be assumed as absolute—no effort will be considered in improving the accuracy of existing datums by extending gage periods. The tidal zoning error can be reduced by lessening the amount of time and range correction needed by establishing more tide stations for use in direct control of the survey.

c. TCARI. Use of NOAA's "Tidal Constituent and Residual Interpolation" (TCARI) procedures can also reduce tidal zoning errors. Project planning an implementation are focused on finding the practical balance between the number of tide stations required and the amount of tidal zoning required. This in turn depends upon the complexities of the tidal characteristics in the area along with the resources and logistics required to establish and maintain tide stations. Calibrated tide gages that are configured and installed to minimize dynamic errors result in the measurement errors usually being minor contributors to the tides error budget. The estimated total tides error can then be root-summed-squared with all of the other hydrographic survey error sources to estimate the total survey error budget.

d. Example tidal zoning project. Even in these larger tidal ranges the gage density appears sufficient to model the MLLW datum variation by interpolation throughout the deep draft portion of the project. Figure E-8 is a graphic showing the CO-OPS discrete tidal zoning scheme for Porthmouth, NH. If RTK procedures were not employed at this project site, time and range correctors for each zone would be applied to an appropriate tide station installed in the harbor to account for time and range changes in the project area. The closest NOAA operating NWLON stations are Boston, MA and Portland, ME.

E-6. Hydrodynamic Tidal Modeling of Navigation Projects.

a. From the above, it would appear that many USACE deep-draft navigations will have a sufficient density of NOAA CO-OPS tidal data such that spatially interpolated models will be adequate. Interpolated models can be:

(1) a linear interpolation of elevation relationships over relatively short distances.

(2) a discrete tidal zoning interpolation based on changes in cotidal lines over the survey area.

(3) a continuous tidal zoning interpolation model such as TCARI.

Where this is not the case, then a hydrodynamic tidal model may have to be generated to define the MLLW datum plane throughout a project.

b. The technical process of developing a hydrodynamic tidal model of a typical coastal inlet, and calibrating that model to one or more fixed gages, is relatively straightforward and models for performing this are well documented in part II of EM 1110-2-1100 (*Coastal Engineering Manual*) and other sources. Many USACE navigation projects have been extensively studied over the years and existing numerical models may be readily utilized to assess the tidal datum relationships—e.g., activities studied under the ERDC/CHL "Diagnostic Modeling System."

c. Projects requiring hydrodynamic tidal modeling to define the MLLW datum can be accomplished by any number of organizations. Some of these include:

(1) District Hydrology & Hydraulics (H&H) sections.

(2) Coastal Engineering A-E firms.

(3) NOAA CSDL VDatum Team.

(4) ERDC/Coastal and Hydraulics Laboratory (CHL).

d. Each of the above options will have different approaches, costs, and turn-around response. Cost estimates for this modeling effort can be obtained from any of these organizations. These costs may include gaging programs which will have to be obtained from NOAA. Actual gage installation can be accomplished via an A-E contract with a coastal engineering firm or through NOAA.
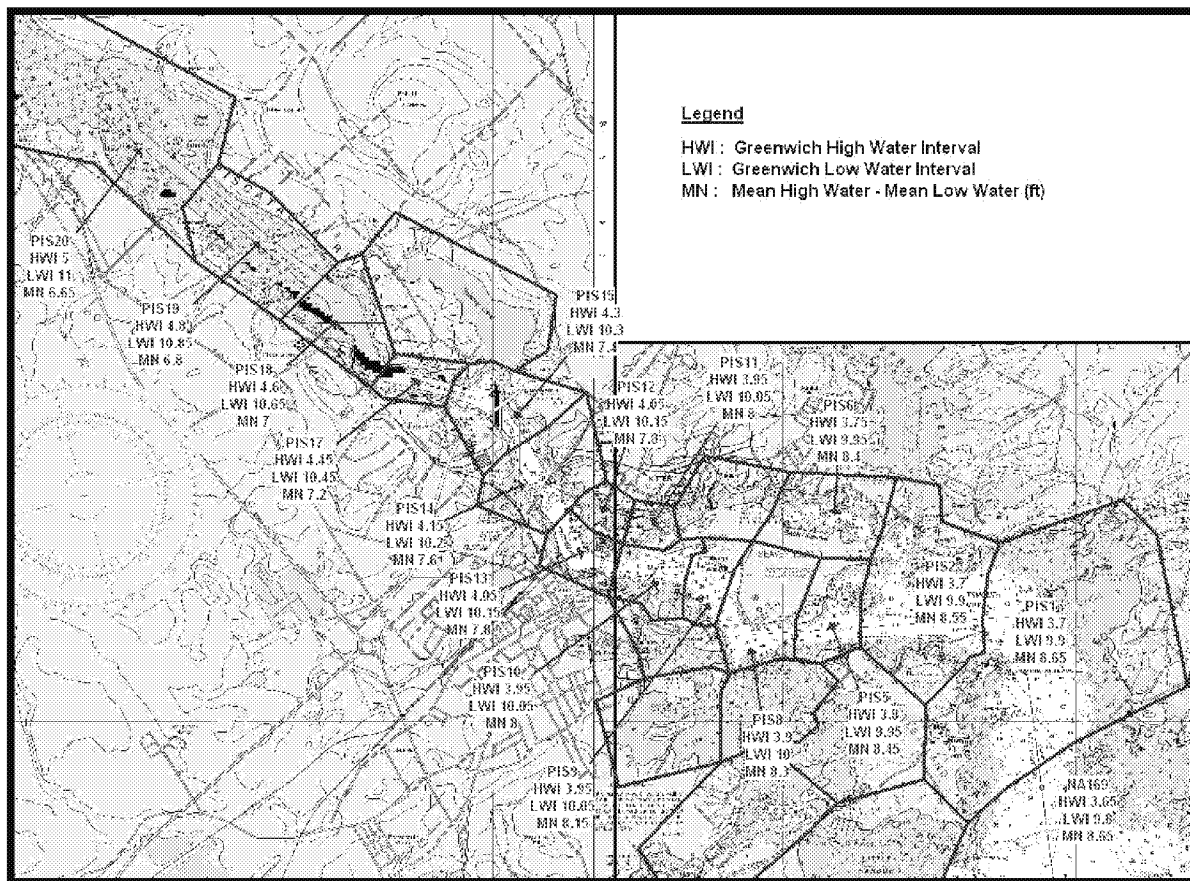
Figure E-8. NOAA discrete tidal zoning scheme for Portsmouth, New Hampshire.

APPENDIX F

Canaveral Harbor, FL— Establishing a PPCP and Tidal Datum Reference when Adequate NOAA Gage Data Exists (Jacksonville District)

F-1. <u>Purpose and Background</u>. This appendix illustrates the establishment of references to the NSRS and the NWLON for a typical navigation project. Figure F-1 depicts a Jacksonville District deep draft project that has been adequately referenced to a NOAA tidal datum and the NSRS. This project supports US Navy, commercial, and cruise ship interests. Underkeel clearances on the US Navy Trident portion of this project are considered critical. A shallow-draft barge canal exists west of the lock at the end of the deep-draft project. That portion of the project illustrates an application where the tidal range is small to negligible and a MSL reference datum is used.
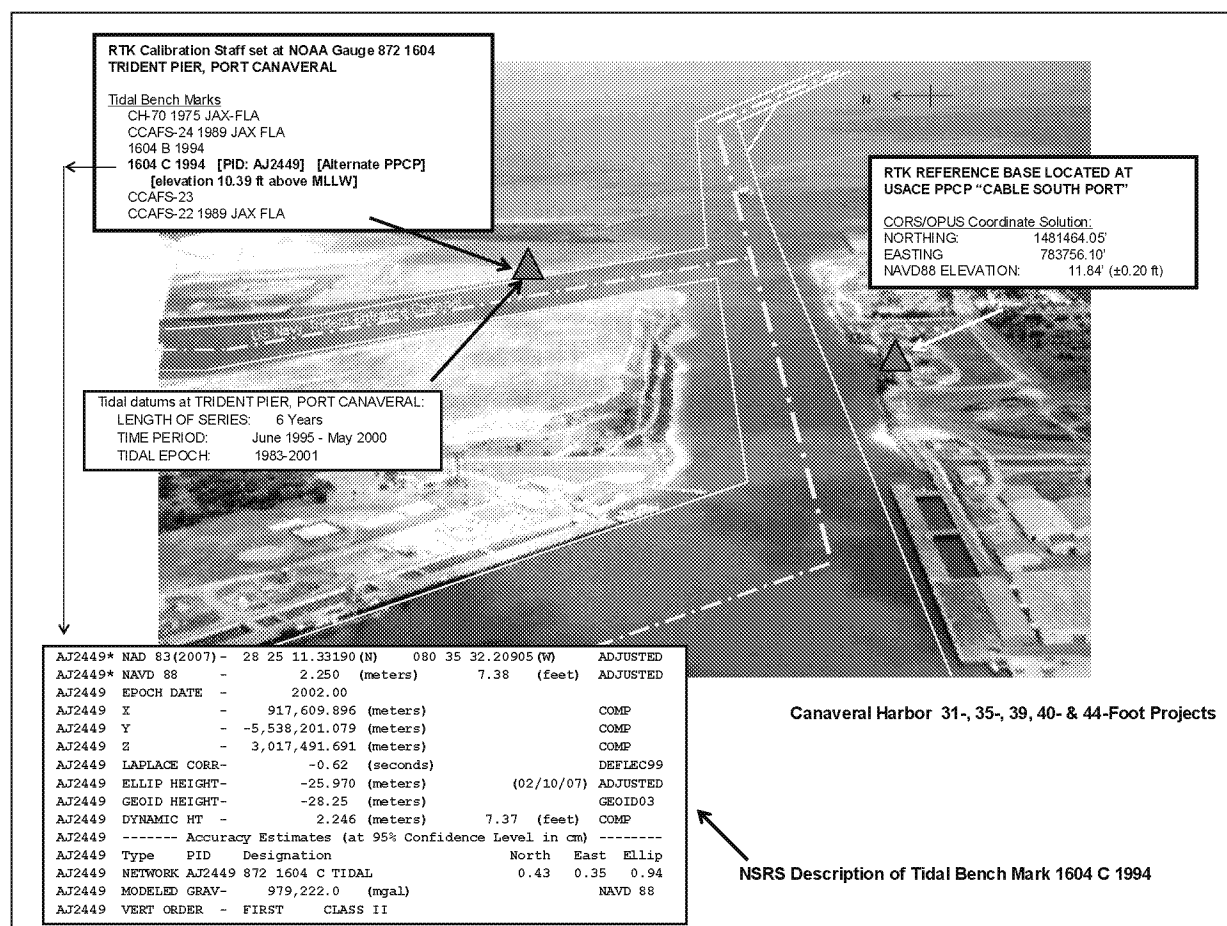
Figure F-1. Tidal PBM and RTK PPCPs established at Canaveral Harbor, FL. (Jacksonville District)

This site contains an active NOAA gage (TRIDENT PIER, PORT CANAVERAL) on the north side of the entrance at the Trident Entrance Channel. This gage has six reference PBMs, one of which (1604 C 1994) is listed in the NSRS with adjusted First-Order NAVD88 vertical control. An existing USACE bench mark (CABLE SOUTH PORT) is located on the south side of the entrance channel. This bench mark had a NGVD29 elevation of uncertain origin and was previously used for tidal corrections at the project. A RTK base station situated on either side of the entrance will provide survey and dredge position coverage to the outer end of the project in the ocean.

F-2. Canaveral Harbor: Deep-Draft Tidal Project.

a. Project description. (See Figures F-2 and F-3). Maintenance of an entrance channel 41 feet deep and 400 feet wide; an inner channel 40 feet and 400 feet wide; a 1200 foot diameter turning basin 39 feet deep; a channel 39 feet deep and 400 feet wide for an 1800 foot length; enlargement of barge channel to 12 feet deep and 125 feet wide to the Intracoastal Waterway; a channel extension 31 feet by 300 feet by 1,500 feet dredged west of turning basin; and a barge lock 90 feet wide and 600 feet long west of the harbor dike; and two entrance jetties to the 12-foot contour. Length of project is about 11.5 miles. The entrance channel and part of the inner channel have been deepened to 44 feet for the Navy's TRIDENT Project.

Figure F-2. Canaveral Harbor project: Trident basin, cruise ship basin, and barge canal lock.
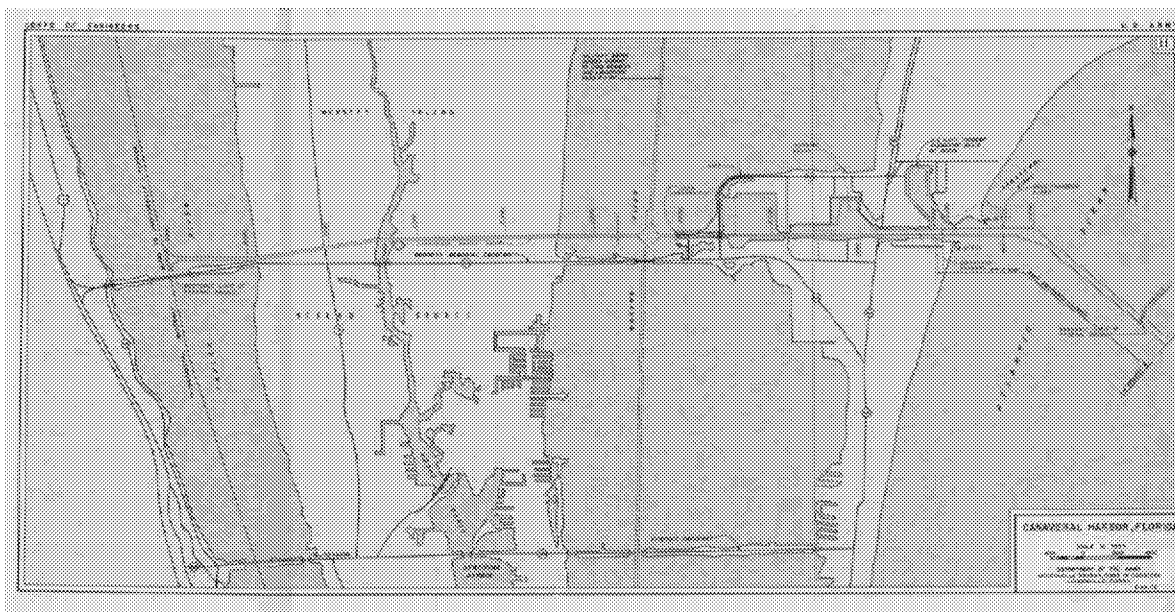
Figure F-3. Canaveral Harbor project map.

Figure F-4. NOAA gage "TRIDENT PIER" in Trident Basin.

b. Tidal datum reference. NOAA CO-OPS data for TRIDENT PIER gage (Figure F-4) indicates it is relatively current with a 6-year recording series from 1995 to 2000. No significant

deepening or entrance modifications have been made since that period. Thus this NOAA gage was deemed as acceptable for the primary datum reference on this project. (Had an existing, or unsuitable, tide station not been located at this project site, then a short-term gage would have had to be installed to determine a reference datum following NOAA CO-OPS standards in "*Computational Techniques for Tidal Datums Handbook*" (NOAA 2003).

(1) Tidal reference bench marks. Three of the published tidal bench marks at the TRIDENT PIER gage were recovered and Third-Order level runs between these marks indicated they were stable internally to within 0.02 ft. Recovery notes on these tidal bench marks were transmitted to NOAA CO-OPS. (As above, had no reference tidal bench marks been recovered, then for all practical purposes, the gage is lost and new tidal observations would be required).

(2) Calibration tide staff. A tide staff was set at the TRIDENT PIER site relative to published MLLW datum on the tidal PBMs. The staff zero was set at MLLW so visual readings were direct elevations of the water surface above MLLW.

c. Primary "PPCP " tidal reference mark. Tidal bench mark "1604 C 1994" at the TRIDENT PIER gage site has a solid First-Order (II) orthometric elevation and observed ellipsoid height observations—see the NSRS extract in Figure F-1. The estimated 95% confidence in the observed ellipsoid height is less than 1 cm. This tidal PBM, with published geodetic, ellipsoidal, and tidal reference elevations, is the obvious choice as the designated PPCP for this project. No additional field survey observations would be needed at this project if this point were used as an RTK base station. Since GPS observations were once made at this mark (per NSRS description), its use as an RTK base is presumed adequate. However, due to site access restrictions at this military site, an alternate RTK base station PBM was established on the south side of the channel at USACE PBM "CABLE SOUTH PORT."
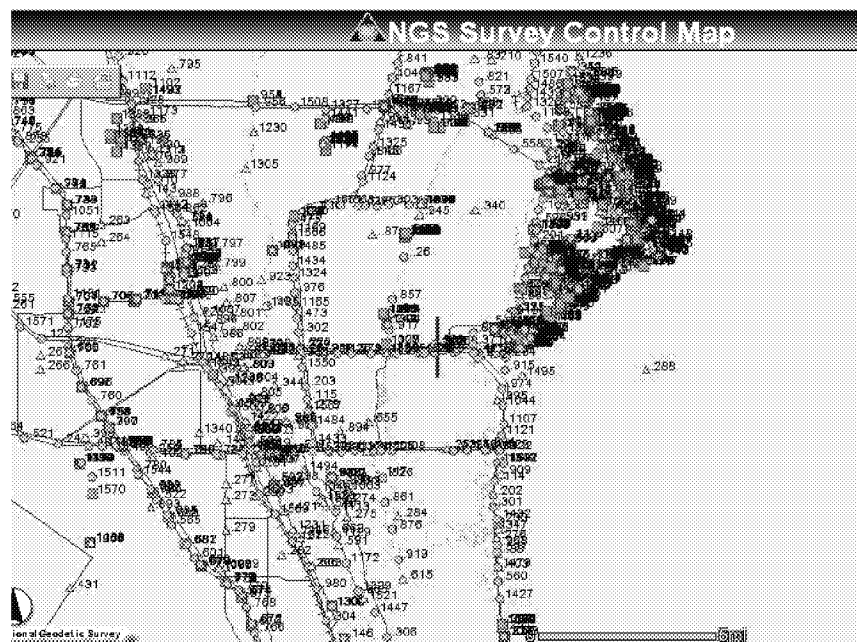
Figure F-5. NGS control network in Cape Canaveral area.

d. GPS surveys to position a primary RTK base station. Obtaining a PPCP for this project was not an issue at this site, given the dense NSRS network in the region—see Figure F-5. USACE PBM " CABLE SOUTH PORT" on the south side of the channel was positioned using CORS/OPUS techniques. An 8-hour session of CORS observations were recorded and transmitted to OPUS for reduction. The results along with descriptive data were transmitted through OPUS-DB for published input to the NSRS. A static baseline was simultaneously observed from tidal PBM 1604 C 1994 as a check on the CORS/OPUS solution. These observations were performed concurrently with hydrographic survey observations so no additional field effort was required to perform these control surveys. PBM "CABLE SOUTH PORT" is thus the designated PPCP for this project.

e. MLLW tidal model. Since VDatum coverage did not exist at this project site in 2009, an estimated tidal model was required. Based on comparisons in diurnal tide ranges between TRIDENT PIER and other NOAA gages in the surrounding offshore region (i.e., ocean pier gages north and south of Canaveral—see Figure F-6), there did not appear to be any significant tidal MLLW datum gradient between the ocean and the interior channels up to the Trident Basin. This may be due to the relatively wide entrance. A constant NAVD88-MLLW difference of 3.01 ft was therefore assumed constant throughout the project. This difference was computed using data at Tidal bench mark "1604 C 1994"—i.e., 10.30 ft NAVD88 - 7.38 ft MLLW. Future VDatum coverage is not expected to significantly modify this constant model since these NOAA gages would likely be used to develop the VDatum model.
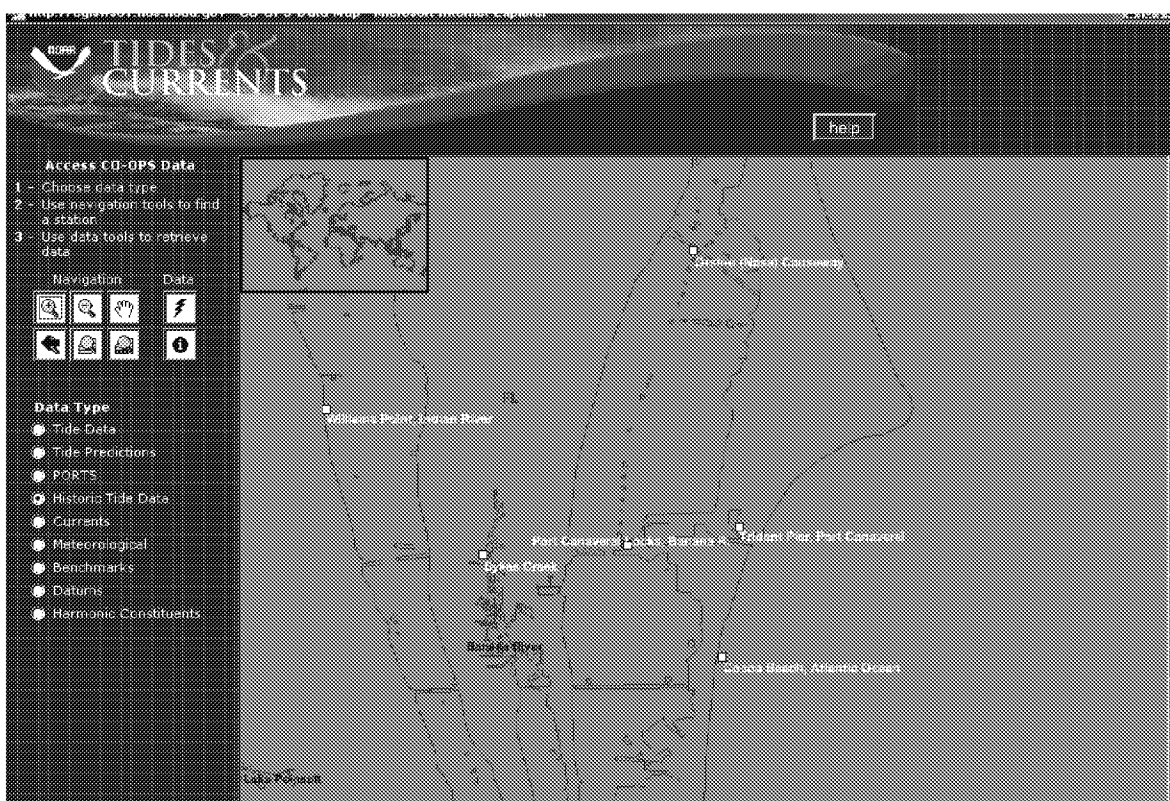
Figure F-6. NOAA tide gage data vicinity of Cape Canaveral.

F-5

f. *Measurement & payment survey procedures.* The RTK base station is initialized at PBM "CABLE SOUTH PORT" using the newly NSRS published coordinates based on CORS/OPUS solutions. RTK observations are adjusted in the vessel positioning software to correct for the NAVD88-MLLW datum difference—i.e., the "K" term referenced Chapter 4. Likewise geoid height variations ("N") over the project are automatically adjusted in the GPS acquisition or processing software.

F-3. <u>Canaveral Lock and Barge Canal to Banana River and Indian River: Non-Tidal</u>. This section illustrates procedures for referencing construction datums in areas with little or imperceptible tide ranges. Tidal influences are small in the areas (Banana River and Indian River) west of the Canaveral Lock, given the nearest inlets north and south are over 20 miles distant. Figure F-7 depicts available tidal information from the Florida Department of Environmental Protection, Land Boundary Information System (LABINS). Survey reaches outside (west of) the Canaveral Lock are depicted as non-tidal, based on LABINS and NOAA gage data. It was decided that dredging elevations within these reaches shall be referred to Mean Sea Level (MSL) as the reference construction datum. Previously, dredging datums in this area were related to MLLW relative to NGVD29, as outlined in the 31 August 2007 CEPD Report for this project:

> *"Water surface elevation measurement tidal and geoid undulation corrections have not been hydrodynamically modeled or calibrated throughout the [Canaveral] project area. Portions of the project area may have not been converted from MLW to MLLW datum. Portions are still on NAD27 horizontal datum—Barge Canal. Currently, water surface elevation corrections for dredging measurement & payment are based on extrapolated staff gage readings set from unmodeled benchmarks that are set from benchmarks of uncertain origin, are not referenced to the NSRS, and are referenced to the superseded NGVD29 datum. Project framework and control documents do not define references or relationships between these benchmarks and NOAA tidal gages or tidal benchmarks."*

> *"NOAA tidal PBM G 215 is used for extrapolating water surfaces in the Barge Canal, however the USACE 8.42 ft elevation differs from NGS published 9.92 ft (NGVD29). An unmodeled constant MLLW datum surface (0.5 ft below NGVD29) is assumed throughout the Barge Canal project area. The source of this corrector is uncertain. The tidal range in this project is small—a constant difference may prove to be valid once a model is developed."*

a. The 2007 CEPD report made the following recommendation regarding the Canaveral Barge Canal:

> *"Tidal range gradients west of the Canaveral Lock should also be assessed and developed if significant, per [HQUSACE] guidance. NOAA gage data needs to be obtained up to the IWW (J—M) to determine the low water datum and whether any significant tidal gradient exists."*

b. This section illustrates corrective actions taken based on the 2007 CEPD recommendation.
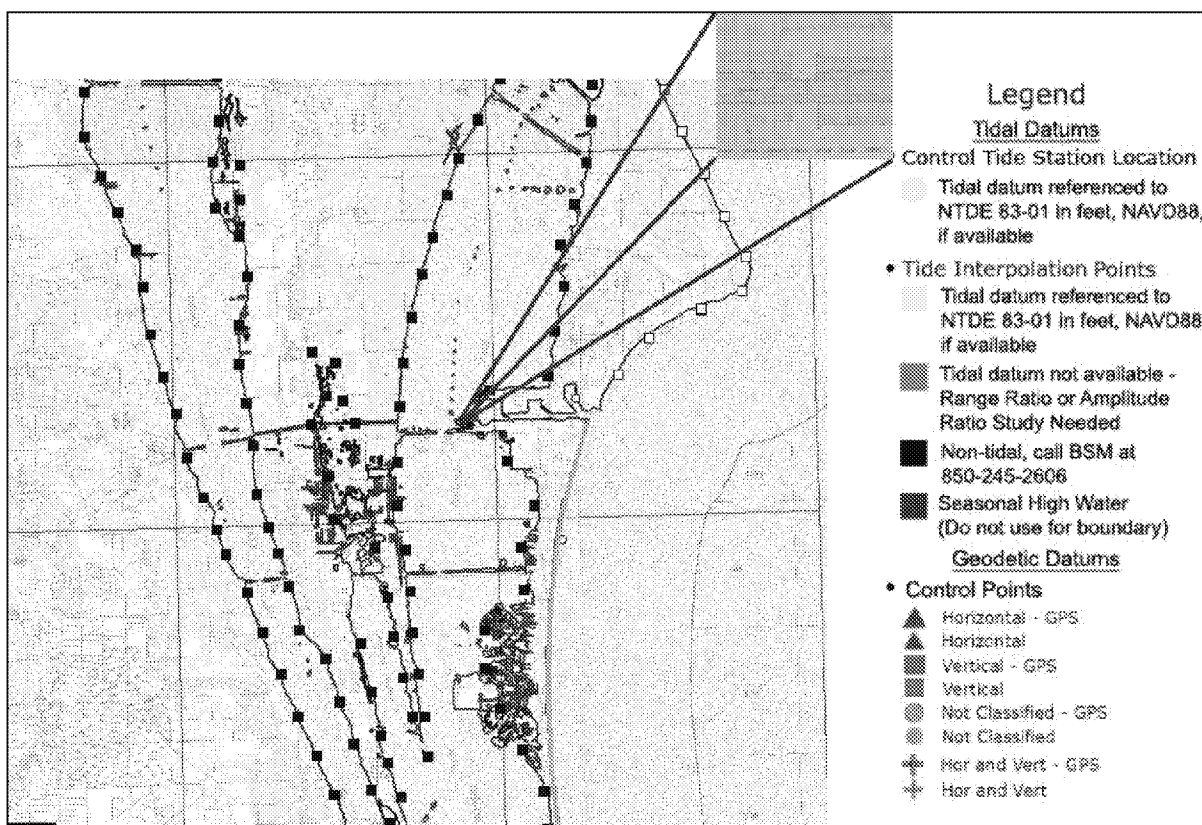
Figure F-7. Canaveral Barge Canal Datum Determination: Non-Tidal gages in Banana River and Indian River Region (Florida Department of Environmental Protection, Land Boundary Information System (LABINS)).

c. The NAVD88-MSL relationship was determined to be 0.70 ft within survey reaches west of the Canaveral Lock chamber (Station 235+00 of the Canaveral Barge Canal). This relationship was determined by performing a distance weighted interpolation from NOAA gages 872-1533 (Orsino Causeway), 872-1456 (Titusville, Indian River), and 872-1456 (Pineda, Indian River)—see Figures F-8 and F-9. Published NOAA gage information at these locations depicts the NAVD88 to MSL relationship—see NOAA datasheets in Figures F-10 and F-11 at the end of this section.

CANAVERAL BARGE CANAL DATUM

NOAA GAGE	NAVD88-MSL ft (x)	DISTANCE MILES (d)	WEIGHT (1/d)	(x) * (1/d)	Weighted Varience	w * (x - x)^2
8721533	0.690	7.414	0.093067	0.064216		0.000028
8721749	0.730	13.717	0.053219	0.038850		0.000027
8721456	0.702	17.41	0.040322	0.028306		0.000001
MEAN (x)	0.707	SUM	0.186607	SUM 0.131372	SUM	0.000056
		weighted mean	0.704		weighted std dev	0.017

Figure F-8. Computation of NAVD88-MSL difference west of Canaveral Lock.

F-7
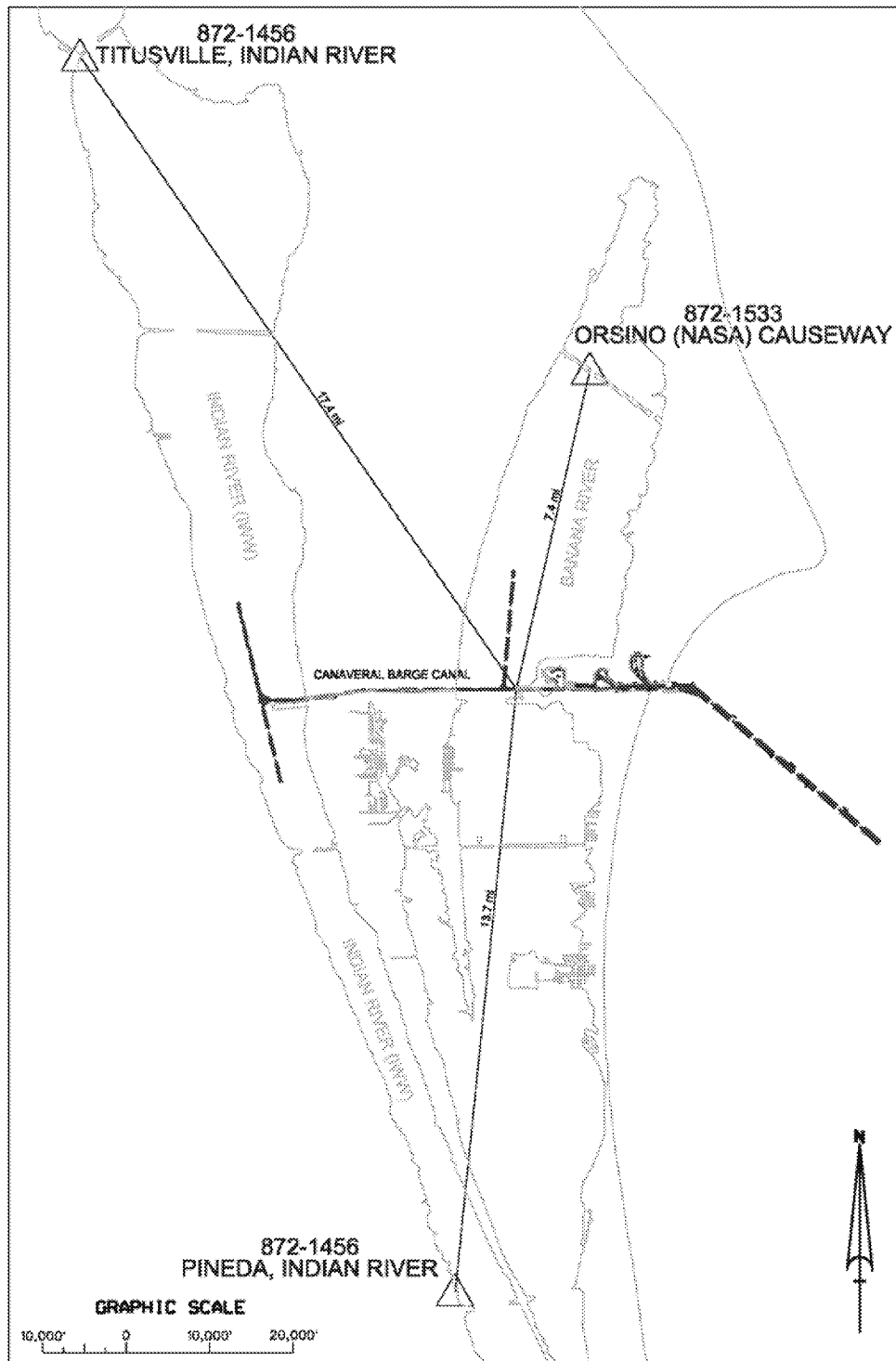
Figure F-9. Spatial interpolation of NAVD88-MSL relationship west of Canaveral Lock.

```
Mar 25 2010 13:17          ELEVATIONS ON STATION DATUM
                          National Ocean Service (NOAA)
Station:  8721456                                   T.M.:        75 W
Name:     TITUSVILLE, INDIAN RIVER, FL              Units:       Feet
Status:   Accepted                                  Epoch:  1983-2001
          Datum        Value   Description
          ---------    -------- -------------------------------------
          MHHW                  Mean Higher-High Water
          MHW                   Mean High Water
          DTL                   Mean Diurnal Tide Level
          MTL                   Mean Tide Level
          MSL          4.01     Mean Sea Level
          MLW                   Mean Low Water
          MLLW                  Mean Lower-Low Water
          GT                    Great Diurnal Range
          MN                    Mean Range of Tide
          DHQ                   Mean Diurnal High Water Inequality
          DLQ                   Mean Diurnal Low  Water Inequality
          HWI                   Greenwich High Water Interval (in Hours)
          LWI                   Greenwich Low  Water Interval (in Hours)
          NAVD         4.71     North American Vertical Datum
          Maximum      5.08     Highest Water Level on Station Datum
          Max Date   19701020   Date Of Highest Water Level
          Max Time     00:00    Time Of Highest Water Level
          Minimum      2.41     Lowest  Water Level on Station Datum
          Min Date   19780204   Date Of Lowest Water Level
          Min Time     00:00    Time Of Lowest Water Level


Mar 25 2010 13:17          ELEVATIONS ON STATION DATUM
                          National Ocean Service (NOAA)
Station:  8721533                                   T.M.:        75 W
Name:     ORSINO (NASA) CAUSEWAY, FL                Units:       Feet
Status:   Accepted                                  Epoch:  1983-2001
          Datum        Value   Description
          ---------    -------- -------------------------------------
          MHHW                  Mean Higher-High Water
          MHW                   Mean High Water
          DTL                   Mean Diurnal Tide Level
          MTL                   Mean Tide Level
          MSL          3.20     Mean Sea Level
          MLW                   Mean Low Water
          MLLW                  Mean Lower-Low Water
          GT                    Great Diurnal Range
          MN                    Mean Range of Tide
          DHQ                   Mean Diurnal High Water Inequality
          DLQ                   Mean Diurnal Low  Water Inequality
          HWI                   Greenwich High Water Interval (in Hours)
          LWI                   Greenwich Low  Water Interval (in Hours)
          NAVD         3.88     North American Vertical Datum
          Maximum               Highest Water Level on Station Datum
          Max Date              Date Of Highest Water Level
          Max Time              Time Of Highest Water Level
          Minimum               Lowest  Water Level on Station Datum
          Min Date              Date Of Lowest Water Level
          Min Time              Time Of Lowest Water Level
```

Figure F-10. NOAA datasheets for tide gages 872 1456 and 872 1533.

```
Mar 25 2010 13:18          ELEVATIONS ON STATION DATUM
                           National Ocean Service (NOAA)

Station:  8721749                                    T.M.:        75 W
Name:     PINEDA, INDIAN RIVER, FL                   Units:       Feet
Status:   Accepted                                   Epoch:   1983-2001


          Datum      Value   Description
          ---------  ------- ----------------------------------------
          MHHW               Mean Higher-High Water
          MHW                Mean High Water
          DTL                Mean Diurnal Tide Level
          MTL                Mean Tide Level
          MSL         2.71   Mean Sea Level
          MLW                Mean Low Water
          MLLW               Mean Lower-Low Water
          GT                 Great Diurnal Range
          MN                 Mean Range of Tide
          DHQ                Mean Diurnal High Water Inequality
          DLQ                Mean Diurnal Low  Water Inequality
          HWI                Greenwich High Water Interval (in Hours)
          LWI                Greenwich Low  Water Interval (in Hours)
          NAVD        3.44   North American Vertical Datum
          Maximum            Highest Water Level on Station Datum
          Max Date           Date Of Highest Water Level
          Max Time           Time Of Highest Water Level
          Minimum            Lowest  Water Level on Station Datum
          Min Date           Date Of Lowest Water Level
          Min Time           Time Of Lowest Water Level

To refer Water Level Heights to a Tidal Datum, apply the desired Datum Value.

Click HERE for further station information including New Epoch products.


To refer Water Level Heights to either
   NGVD (National Geodetic Vertical Datum of 1929) or
   NAVD (North American  Vertical Datum of 1988), apply the values located at:
   National Geodetic Survey
```

Figure F-11. NOAA datasheet for tide gage 872 1749.

F-4. <u>RTK Coverage</u>. Figure F-12 depicts proposed RTK coverage for this projects. Base stations would be located at CABLE SOUTH PORT and CBC-101. The published NSRS datasheet for CABLE SOUTH PORT is shown on Figure F-13.
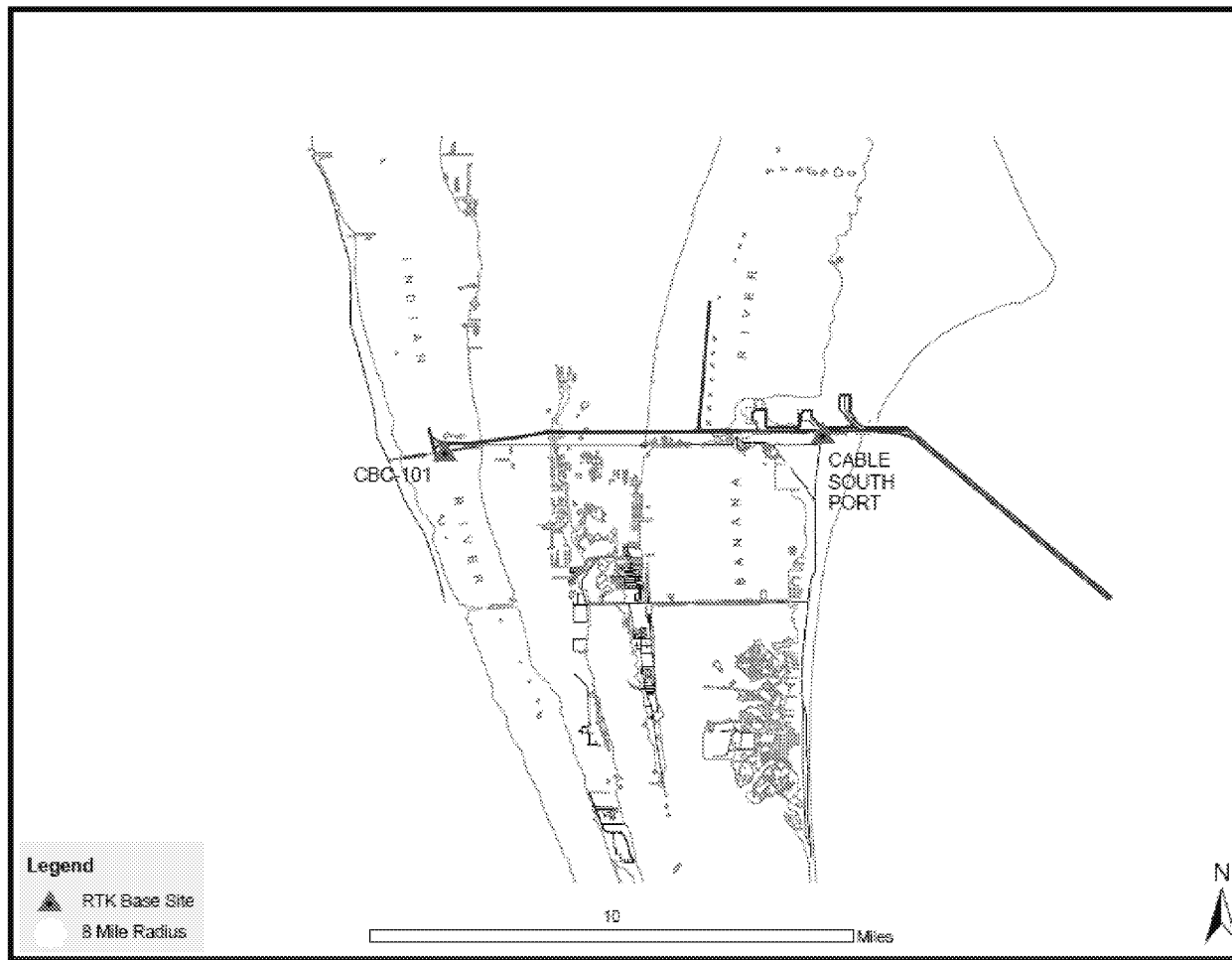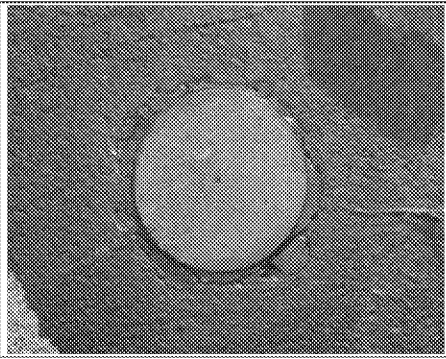

Figure F-12. RTK scheme for Canaveral Harbor.

SURVEY DATASHEET (Version 1.0)

PID: BBBV18
Designation: CABLE SOUTH PORT
Stamping: CABLE SOUTH PORT 1988
Stability: Monument will probably hold position well
Setting: Retaining wall or concrete ledge
Description: From the intersection of I-95 and FL-528, proceed East on FL-528 for 12.6 miles to Astronaut BLVD/A1A S. Proceed South on A1A for 0.6 miles. Take Port Canaveral exit and drive 0.9 miles to N. Atlantic BLVD. Turn left into Port Entrance. Proceed through Port security and take first right onto Herring St. Drive 0.2 miles and turn right onto Glen Cheek Dr. Proceed approximately 0.2 miles to mark on left. Mark is located on a seawall approximately 65 feet east of the centerline of Jetty Park Rd.
Observed: 2010-03-30T16:35:00Z
Source: OPUS - page5 0909.08

Close-up View

REF_FRAME: NAD_83(CORS96)	EPOCH: 2002.0000	SOURCE: NAVD88 (Computed using GEOID09)	UNITS: m	SET PROFILE	DETAILS

LAT: 28° 24' 30.91334"	± 0.018 m		
LON: -80° 36' 11.16570"	± 0.020 m	**UTM** 17	**SPC** 901(FL E.)
ELL HT: -24.601	± 0.043 m	**NORTHING:** 3142529.061m	451551.147m
X: 916660.763	± 0.019 m	**EASTING:** 538876.040m	238889.309m
Y: -5538959.611	± 0.033 m	**CONVERGENCE:** 0.18882919°	0.18882919°
Z: 3016397.958	± 0.035 m	**POINT SCALE:** 0.99961865	0.99995983
ORTHO HT: 3.639	± 0.046 m	**COMBINED FACTOR:** 0.99962251	0.99996370

Figure F-13. Portion of NSRS Datasheet for USACE PPCP "CABLE SOUTH PORT."

APPENDIX G

Fort Fisher Shore Protection and Beach Stabilization Project (Wilmington District)

G-1. Purpose. This appendix contains an example of a Wilmington District project that has been adequately referenced to the current NSRS orthometric datum and to the local tidal datum. The project consists of a 3,000 ft stone revetment sector that has an established reference baseline with local control on a legacy NGVD29 vertical datum. Beach monitoring surveys are performed in an area some two miles to the south of the revetment area, as shown on Figure G-1. The mean range of tide is 4.2 ft and the maximum known storm tide is 10.7 feet above MSL. Average annual shoreline retreat rate is approximately 15 ft in this area.
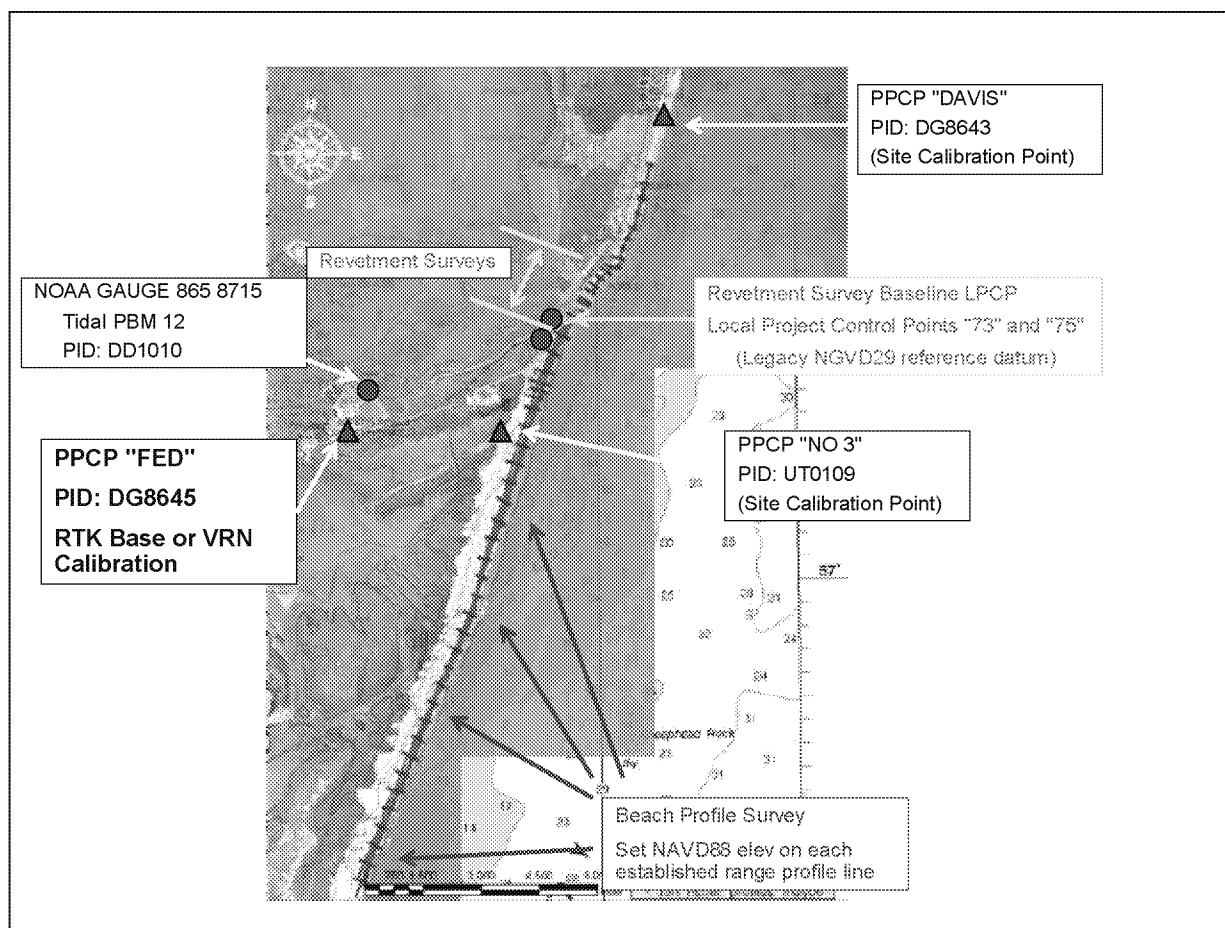

Figure G-1. Fort Fisher revetment and beach monitoring survey scheme.

G-2. Connections to NSRS and Tidal Datum References. The PPCP established for the project is PBM "FED," as shown on Figure G-1. This point is published in the NSRS and has observed NAD83/GRS80 ellipsoid height observations. Excerpts taken from the NOAA NGS published datasheet are shown in Figure G-2.

```
DG8645  *****************************************************************************
DG8645  DESIGNATION -  FED
DG8645  PID         -  DG8645
DG8645  STATE/COUNTY-  NC/NEW HANOVER
DG8645  USGS QUAD    -  KURE BEACH (1979)
DG8645
DG8645                         *CURRENT SURVEY CONTROL
DG8645
DG8645* NAD 83(2007)-  33 57 37.24132(N)      077 56 28.77768(W)      ADJUSTED
DG8645* NAVD 88      -             5.992  (meters)      19.66   (feet)  ADJUSTED
DG8645
DG8645  EPOCH DATE   -          2002.00
DG8645  X            -   1,106,339.885  (meters)                       COMP
DG8645  Y            -  -5,178,836.890  (meters)                       COMP
DG8645  Z            -   3,542,781.497  (meters)                       COMP
DG8645  LAPLACE CORR-           -4.76   (seconds)                      DEFLEC99
DG8645  ELLIP HEIGHT-          -31.476  (meters)          (02/10/07) ADJUSTED
DG8645  GEOID HEIGHT-          -37.37   (meters)                       GEOID03
DG8645  DYNAMIC HT  -            5.986  (meters)      19.64   (feet)  COMP
DG8645
DG8645  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
DG8645  Type    PID    Designation                      North   East  Ellip
DG8645  ----------------------------------------------------------------
DG8645  NETWORK DG8645 FED                               0.57   0.51   1.53
DG8645  ----------------------------------------------------------------
DG8645  MODELED GRAV-     979,620.0    (mgal)                      NAVD 88
DG8645
DG8645  VERT ORDER  -  SECOND     CLASS II
```

Figure G-2. Portion of NGS datasheet for bench mark FED.

The 19.66 ft NAVD88 elevation of FED is based on adjusted leveling observations. The estimated 95% confidence of the ellipsoid height is less than 2 cm. Thus, this is an excellent point for use as an RTK base station from which all supplemental surveys can be referenced.

     a. PPCP verification. The coordinates of PPCP "FED" were site calibrated against three nearby published bench marks in the NSRS—DAVIS, NO 3, and 865 8715 TIDAL 12. (DAVIS for horizontal only). These site calibration checks were based on RTK observations with the base station at FED. (Subsequent surveys used the North Carolina RTN rather than a RTK base at FED). Figure G-3 shows a typical field site calibration at PBM "NO 3." The 0.1 ft horizontal difference and 0.01 ft elevation difference are within acceptable measurement tolerances.
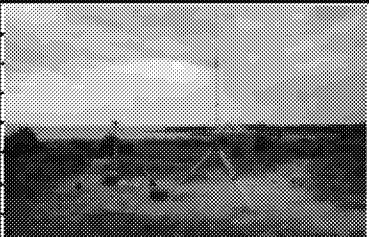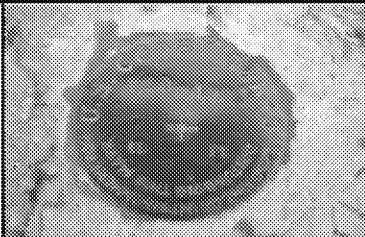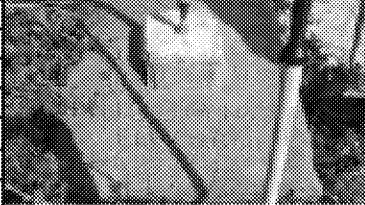
geodynamics
COMPLEX COASTAL CHANGE MADE CLEAR

RTK-GPS Pre-Survey Site Calibration

General

Date	5/3/2005
Project	USACE Fort Fisher Beach Profiles
Surveyor(s)	Freeman / Bernstein
Equipment	Trimble 5700 Basestation, Trimmark III 25 watt RTK Radio, Maxrad 5dB gain Antenna, Zepher Geodetic base antenna, Trimble 5700 RTK rover, Zepher antenna
Weather	Sunny, Few Clouds, 83 F, WNW Wind 15-20 kts.
Units	Meters
Notes	Tidal No 2- destroyed, FF 1-not found, Assembly - not found, FF3 found bubt not good for GPS Observations at the time
Coordinate System	NC State Plane, NAD83 (horiz), NAVD88 (vert)

Basestation Information

Designation	FED
PID	DG8645
Agency	US Coast Guard
Horiz Order	1
Vert Order	2
N	23857.459
E	707461.906
Z	5.992

Datasheet WWW Link	

Benchmark Checks

Designation	NO 3
PID	UT0109
Agency	US BM 1911
Horiz Order	2
Vert Order	2

	Recorded	Published	Difference
N	23869.183	23869.135	-0.048
E	707419.36	707419.549	0.189
Z	1.793	1.804	0.011

Notes	

Figure G-3. Site calibration RTK observations at PBMs "FED" and "NO 3."
(Checks to other site calibration points were made but are not shown in this figure)

G-3

b. Local PBM and TBM control. Figure G-4 lists the local reference PBMs and TBMs that were connected to the PPCP for the project using RTK positioning methods and differential leveling. The legacy NGVD29 elevation was retained for the two reference points used for the stone revetment monitoring survey area (USACE "73" and USACE "75"). The beach monitoring TBMs for each profile range were positioned using total station and RTK methods, and set relative to previously established monitoring range locations. Differential levels were also run through these points and the leveled elevations were held over the RTK elevations. All elevation measurements were relative to PPCP "FED" on the NAVD88 reference datum.

NGS Mon "FED", Stamped "FED 1993"
    NAD '83 (2007)
        N: 78, 271.26'
        E: 2,321,065.58'
    Elevation: 19.66' (adjusted) NAVD"88

NGS data sheet attached, below

Rock Revetment Survey based on (NGVD'29):

Point	Baseline Station	Northing	Easting	Elevation	Description
73	568+91.19, -225	81,832.0151	2,328,078.783	15.16'	USACOE Monument
75	566+38.80, -200	82,051.6653	2,328,251.723	21.40'	USACOE Monument

Beach Profile Temporary Survey Control, NAVD'88

Point	Northing	Easting	Elevation	Description
1005	83,131.9915'	2,329,171.5304'	8.93'	18" WOOD STAKE
1006	83,637.4830'	2,329,405.8200'	12.82'	18" WOOD STAKE
1007	84,023.9970'	2,329,509.8780'	13.14'	18" WOOD STAKE
1008	84,417.9320'	2,329,587.6850'	14.33'	18" WOOD STAKE
1009	84,807.9160'	2,329,677.6160'	14.91'	18" WOOD STAKE
1010	85,292.8280'	2,329,800.0250'	15.17'	18" WOOD STAKE
1011	85,378.4740'	2,329,879.1460'	15.36'	18" WOOD STAKE
1014	81,278.5780'	2,328,047.0970'	9.50'	18" WOOD STAKE
1019	79,480.3100'	2,326,830.2700'	9.04'	18" WOOD STAKE
1023	82,644.5270'	2,328,815.5440'	10.86'	18" WOOD STAKE
1024	82,014.4670'	2,328,323.9140'	22.59'	40D NAIL
1025	80,118.1730'	2,327,103.5670'	19.01'	18" WOOD STAKE
4010	82,016.5763'	2,328,322.6362'	23.52'	18" WOOD STAKE
4011	81,297.8452'	2,328,013.0360'	9.60'	18" WOOD STAKE
4105	76,622.5126'	2,326,039.5377'	16.82'	18" WOOD STAKE
4109	77,659.4700'	2,326,346.0500'	12.45'	18" WOOD STAKE
4121	75,134.2843'	2,325,579.0767'	13.44'	18" WOOD STAKE
4125	74,193.2775'	2,325,347.2231'	11.84'	18" WOOD STAKE
4130	73,268.3720'	2,324,986.0883'	15.12'	18" WOOD STAKE
4134	72,341.6740'	2,324,640.1954'	13.26'	18" WOOD STAKE
4143	70,966.8913'	2,324,028.6202'	10.84'	18" WOOD STAKE
4148	70,059.4533'	2,323,607.9422'	12.62'	18" WOOD STAKE
4442	70,805.7718'	2,323,964.7684'	7.52'	18" WOOD STAKE
4450	78,271.1098'	2,321,065.5552'	19.44'	18" WOOD STAKE
4459	70,805.6693'	2,323,964.8716'	7.47'	18" WOOD STAKE
4616	70,805.7390'	2,323,964.8300'	7.37'	18" WOOD STAKE
8097	82,729.9630'	2,329,006.9098'	10.44'	18" WOOD STAKE
8331	79,534.2030'	2,326,908.7685'	14.60'	18" WOOD STAKE
25018	83,377.3410'	2,329,272.9640'	8.47'	18" WOOD STAKE

Figure G-4. Primary, local, and baseline range control for Fort Fisher project.

c. *Conversion from NAVD88 to NGVD29.* The survey specifications required that range profile data be referenced to NGVD29 in order to compare prior surveys on that legacy datum. All topographic and hydrographic surveys of each profile range observed on NAVD88 were converted to NGVD29 based on an average VERTCON difference of (-) 0.956 ft (NAVD88 - NGVD29). This conversion assumed prior surveys referenced to NGVD29 were based on published NSRS connections. (It would have been preferable to establish NAVD88 on an existing mark with a published agency elevation to derive the conversion value.)

d. *Tidal reference datum.* The tidal relationship to NAVD88 was estimated using the nearest NOAA gage to the project site. Figure G-5 shows a number of NOAA gages in the region; however, only the Wilmington Beach gage on the coast is most representative of the project site to the south. The Southport gage to the south has similar mean and diurnal tide ranges as the Wilmington Beach gage. Therefore, the interpolated tidal characteristics at the project site are likely representative of these gages.
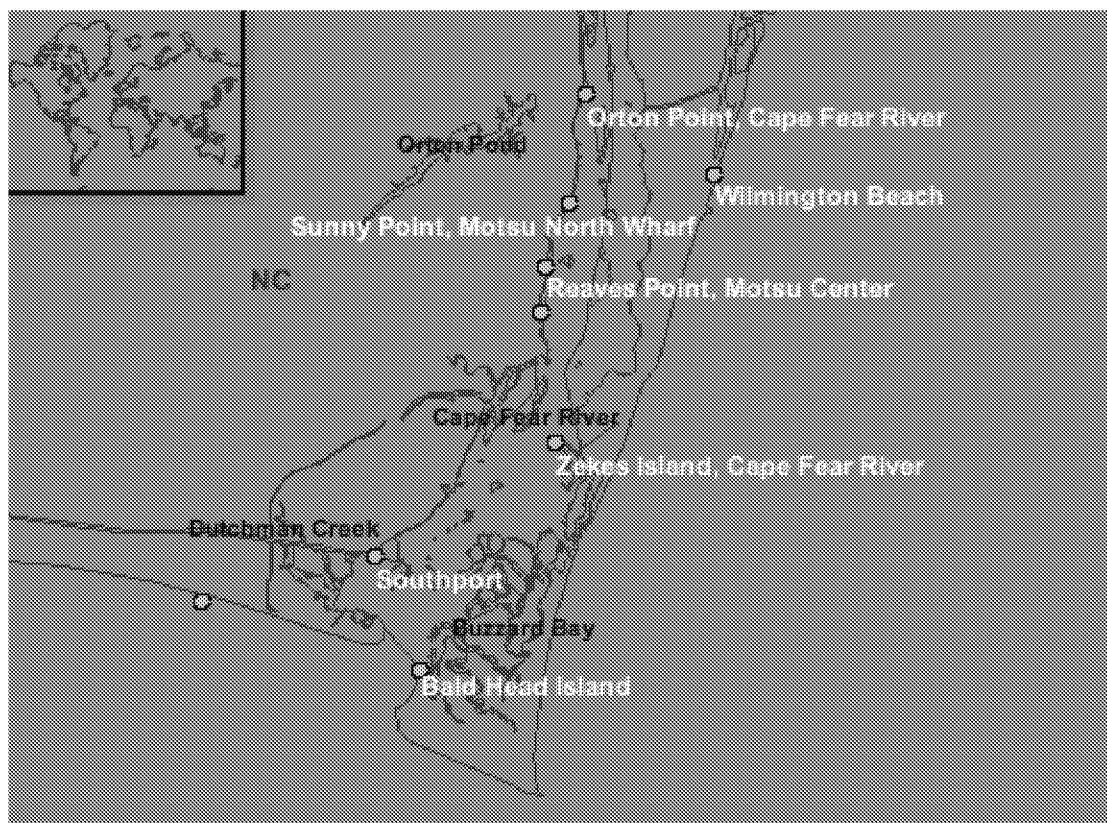

Figure G-5. Published NOAA/CO-OPS gage data in vicinity of Fort Fisher, NC.

(1) The NOAA CO-OPS station tabulation for this gage is shown in Figure G-6.

```
Tidal datums at WILMINGTON BEACH [865 8559] based on:

      LENGTH OF SERIES:      4 MONTHS
      TIME PERIOD:           March 1977 - June 1977
      TIDAL EPOCH:           1983-2001
      CONTROL TIDE STATION:  8658120 WILMINGTON, CAPE FEAR RIVER


Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

      MEAN HIGHER HIGH WATER (MHHW)                 =  1.433   [4.70 ft]
      MEAN HIGH WATER (MHW)                         =  1.329
      NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)     =  0.902   [2.96 ft]
      MEAN TIDE LEVEL (MTL)                         =  0.688
      MEAN SEA LEVEL (MSL)                          =  0.686   [2.25 ft]
      MEAN LOW WATER (MLW)                          =  0.047
      MEAN LOWER LOW WATER (MLLW)                   =  0.000
```

Figure G-6. NOAA tide gage Wilmington Beach.

(2) Based on the data in Figure G-6, NAVD88 is 0.71 ft above MSL (0.902 m – 0.686 m). The difference between NGVD29 and MSL is then 0.25 ft (0.96 ft – 0.71 ft). The difference between NAVD88 and MSL is assumed the same at the project site to the south. (A VDatum model may show slight variations).

(3) The gage at NOAA historical tide station 865 8715 is no longer published by NOAA CO-OPS. This gage would not be relevant to this project given its interior location.

(4) Based on the above data, the geodetic and tidal relationships at PPCP "FED" could be tabulated as shown in Table G-1.

Table G-1. Elevations at PPCP "FED."

Datum [1]	Elevation	Referenced From	Estimated Uncertainty	Relative to
MLLW	22.62 ft	NOAA gage 865 8559	±0.2 ft	NWLON
NGVD29	20.62 ft	VERTCON transform	±0.3 ft	NSRS
MSL	20.37 ft	NOAA gage 865 8559	±0.2 ft	NWLON
NAVD88	19.66 ft	NSRS	±0.1 ft [2]	NSRS
MHW	17.92 ft	NOAA gage 865 8559	±0.2 ft	NWLON
Ellipsoid	-103.27 ft	GPS observations	±0.05 ft	NAD83/GRS80

[1] Tidal datum elevations are estimated based on NOAA gage 865 8559 (Wilmington Beach).

[2] Uncertainty relative to NSRS not factored in to supplemental surveys.

APPENDIX H

East Branch Clarion River Dam and Spillway Control Surveys (Pittsburgh District)

H-1. Introduction. This appendix is an example of establishing NSRS control on a Pittsburgh District dam and reservoir on the East Branch of the Clarion River (Figure H-1). A single primary PBM was connected by a combination of GPS and CORS observations from surrounding NSRS bench marks. Secondary deformation monitoring points were controlled from the primary PBM and dam and spillway profile surveys were run. Elevation data sets developed from LIDAR collected in 2006 was then used for comparative analysis against the profile surveys.

a. This appendix was compiled from survey reports by two Pittsburgh District Contractors: TerraSurv, Inc. and Photo Science, Inc. TerraSurv performed the initial NSRS network connections in May 2008 and Photo Science performed comparative data mapping analysis in 2009.

Figure H-1. East Branch Dam and Reservoir (Elk County, PA).

b. The Pittsburgh District issued task orders to TerraSurv and Photo Science to establish a positional relationship/correlation between the hydraulic, geodetic, and engineering design datum at the East Branch Clarion River Dam and Spillway located in Elk County, PA. This

H-1

work supports the District's adherence to ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*), namely to address the "… need to firmly establish the relationships between hydraulic and geodetic datums…"

H-2. <u>Project Location</u>. Authorized by the Flood Control Act of 1944, East Branch Clarion River Lake is one of 16 projects in the USACE, Pittsburgh District. An important link in a system of flood risk management projects, East Branch provides flood protection for the Clarion River Valley as well as the lower Allegheny and upper Ohio Rivers. Completed in 1952, East Branch Lake has the capability to store the equivalent run-off of 21.84 inches of precipitation from its 72.4 square mile drainage area.

H-3. <u>Scope of Work</u>. The initial requirements outlined for the project were as follows:

a. Establish a primary NSRS bench mark and two secondary bench marks at each project location. This is intended to conform to the criteria in ER 1110-2-8160 that "...the designed, constructed, and maintained elevation grades of projects shall be reliably and accurately referenced to a consistent nationwide framework, or vertical datum—i.e., NSRS..."

b. Determine validity of dam/spillway design grade to current as-built surveys and other sources (LIDAR mapping).

c. Establish relationship/correlation between hydraulic and geodetic datums at each project.

H-4. <u>CEPD Assessment</u>.

a. As part of the 2007 Corps-wide CEPD review, the following actions were taken to accomplish the above objectives.

(1) Perform reconnaissance surveys at the dam site to verify existing local control.

(2) Develop recommendations for Corrective Actions.

(3) Establish NSRS Project Control, e.g., Primary bench mark and two secondary PBMs.

(4) Survey gage reference points.

(a) Pool.

(b) Outflow.

(5) Establish pool elevation relative to NAVD88.

(6) Profile dam and spillway.

(7) Reference deformation monitoring points to NAVD88 and PPCP.

(8) Perform alignment measurements.

b. A CEPD research of existing control data at the East Branch Dam site indicated that geodetic control was referenced to legacy datums of dated origins. The CEPD review is summarized below.

"East Branch Dam horizontal positions are controlled by traverses tied to USGS Stations TT3K, TT6K, and TT7K, datum uncertain, and are computed on Pa. North-Zone System of Coordinates. Elevations are based on those same USGS Stations. Topography was compiled by plane table in 1946 and traced on Map Sheet 038b-U1-16/1 through 10. Additional topography was compiled from aerial photographs exposed November 1979 and consists of Map Sheets 038b-U1-101/1 thru 4, scale 1:2,400, control based on N.A.D. 1927 and N.G.V.D. 1929."

H-5. Options Considered for Corrective Action Field Surveys.

a. The following methods were considered for connecting the Primary Project Control Point (PPCP) to the NSRS.

(1) Differential Leveling (Orthometric Height Accuracies ~ 0.5-2 cm).

(2) GPS Network-"Blue Booking" (Orthometric Height Accuracies ~ 2-3 cm).

(3) OPUS DB-Using CORS Network (Orthometric Height Accuracies ~ 5-10 cm).

b. Differential leveling options.

(1) Labor Intensive.

(2) High cost.

(3) Projects are in isolated areas, some are quite distant from existing level lines.

(4) Horizontal positions not determined.

(5) Determined to be not economically feasible.

c. Blue Book option.

(1) Create a GPS network, format and submit to NGS.

(2) Advantages:

(a) Ties to adjacent points and bench marks.

(b) Multiple occupations.

(c) Homogenous network.

(3) Disadvantages:

(a) More complex to implement.

(b) Requires multiple receivers and planned GPS campaign.

(c) Costly data processing.

d. OPUS-DB option.

(1) Advantage: Relatively simple to implement (Single GPS receiver).

(2) Disadvantages:

(a) No ties to bench marks or adjacent points.

(b) Single occupation.

(c) Requires minimum of two 4-hour occupations.

e. The Blue Book method was selected for the following reasons.

(1) Provides high quality control at each project, as requested by Project Managers.

(2) Utilizes ties to bench marks/HARN/CORS.

(3) Takes advantage of GPS data collected at each site 2005-2008 (i.e. less than 4 hour sessions, not acceptable to OPUS-DB).

H-6. Recommended Primary Control Bench Mark at Project Site. It was recommended that USACE mark "1-500" be used as the primary project bench mark, and M1 (right bank, on dam axis) and M2 (left bank, dam axis) be used as secondary bench marks. Yearly ties are made between these three marks during the alignment survey. Levels to the water gage reference marks could be run from any of the aforementioned marks. A precise level tie has been recently run to PBM 1-500 from an NSRS mark. This data could be used to submit a vertical blue book project to the NSRS. This will require that the raw data file be retrieved, and that a differential level tie be made from Z 337 to another mark on the same line (two mark tie). It is believed that an adjacent mark within a reasonable distance should not be hard to find. The initial evaluation assumed that PBM 1-500 would be obstructed and not suitable for GPS. Two options for GPS derived elevations were considered in the initial evaluation. One option was to simultaneously occupy a secondary project bench mark, M2, and two nearby NSRS bench marks. The primary PBM 1-500 is partially obstructed and not suitable for GPS. Levels are run yearly between M1, M2, and 1-500 so there would be sufficient data available to provide an accurate tie to PBM 1-

500. Alternatively, 1-500 or M2 could be occupied for two sessions of at least 240 minutes (4 hours) and submitted to OPUS-DB.

H-7. <u>Primary NSRS Control Network</u>. Existing USACE survey disk (PBM) "1-500" is the designated primary control point for this project. This disk is located atop the upstream parapet wall at the land abutment for the concrete bridge leading to the intake tower on the right bank of the reservoir. This abutment is not rigidly connected to the bridge; rather the bridge sits on the abutment seat. This mark is shown in Figure H-2.
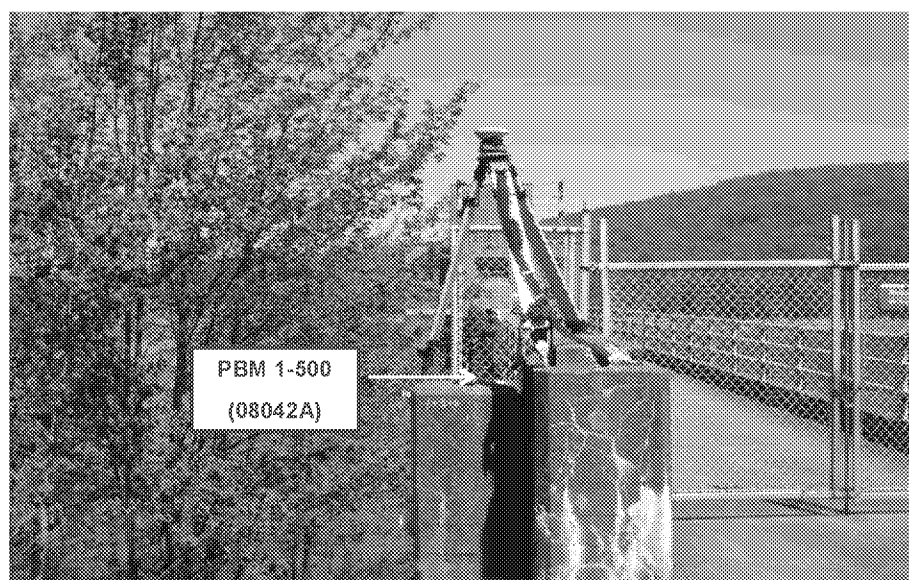

Figure H-2. Primary Project Control Point 1-500 on bridge leading to intake tower.

a. Several options were available for bringing control in from the National Spatial Reference System (NSRS) to the project. A search was made of the NSRS database. There is a level line running north-south along a railroad located approximately 1.6 air miles west of the dam. Bench mark Z 337 (MA0592), located on a bridge abutment, was recovered on this line, and determined to be suitable for GPS observations. Research revealed that a survey crew from the Corps of Engineers had run a line of differential levels from this bench mark to PBM 1-500 and back in 2003 using a Zeiss DiNi 12 digital level and 2 m invar rods. However, only field notes were found, the raw data (which could be Blue Booked) was not found. A search was then made of the NSRS for bench marks located on stable structures that also have HARN horizontal positions published on NAD83 (NSRS2007). This search returned two marks listed Table H-1.

Table H-1. HARN Bench Marks.

Name of Mark	PID	Horizontal Accuracy	Vertical Order	Location	Setting
TTS 64 K	MA0735	0.3 cm	II-Class 0	17.9 mi west	Bedrock
V 25	MA0095	0.3 cm	II-class 0	19.9 mi east	Bridge abutment

b. All three NSRS marks are located on stable structures, and are shown on the map in Figure H-3. The data from these three marks to the on site primary control point 1-500 was formatted for Blue Book submittal to the NSRS. This resulted in the inclusion of PBM 1-500 in the NSRS database. The horizontal datum is NAD83 (CORS1996), obtained via an OPUS solution. The vertical datum is the NAVD88, obtained via direct GPS ties to the three bench marks described above. The three marks located at the dam, 1-500, M1, and M2, were connected via differential levels, static GPS, and EDM/angle measurements. This data was not Blue Booked.
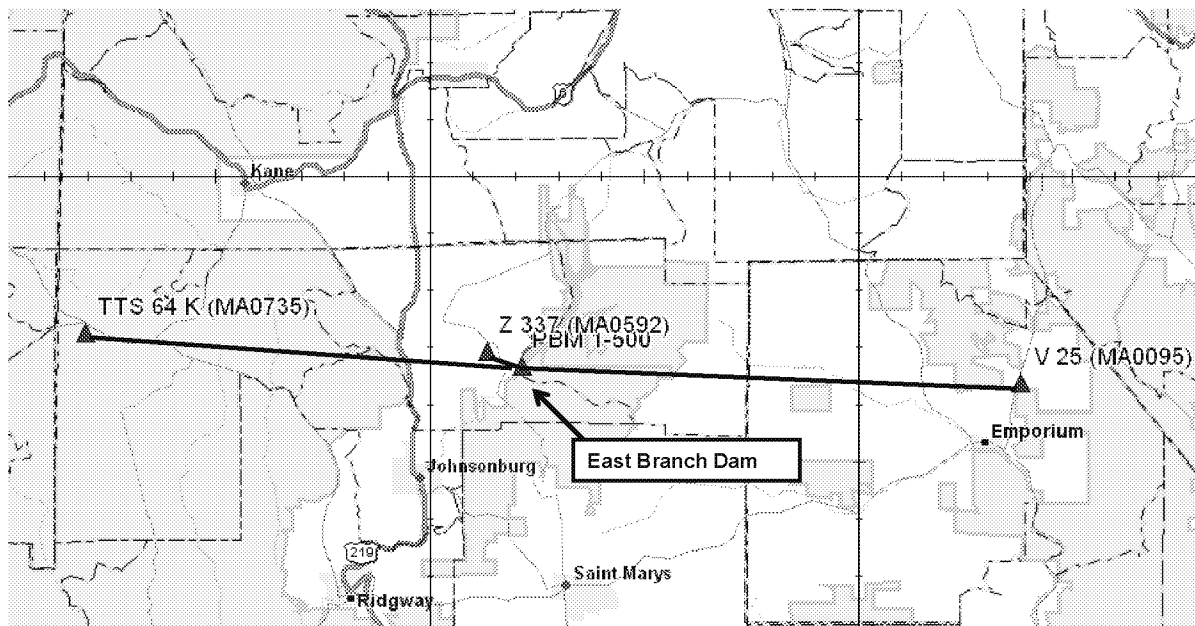

Figure H-3. NSRS control scheme for establishing elevation on PBM 1-500.

H-8. GPS Survey Procedures to Connect PBM 1-500. Three Trimble dual frequency receivers (a 5700 and two R8 GNSS receivers) were used on days 133 and 134 of 2008. Fixed height tripods were utilized for the occupations of the NSRS bench marks. A standard survey tripod with tribrach was used at PBM 1-500 due to the difficulty of using a fixed height tripod at that location. Each point had two independent occupations. The Trimble 5700 receiver with a Zephyr antenna was setup on both days on PBM 1-500, and collected data during the entire day. The two HARN/ bench marks (V 25 and TTS 64 K) were each occupied twice, once on day 133 and once on day 134, each time with a different operator/tripod/receiver. The nearby bench mark, Z 337, was occupied twice on day 134, at different times of the day. The data collected at PBM 1-500 (GPS# 08042A) was submitted to the Online Positioning User Service (OPUS). The OPUS online processor selected three nearby CORS and determined the position of the submitted point. The results are shown in Table H-2.

Table H-2. OPUS Solutions.

Day	Duration Minutes	CORS USED	Overall RMS	% OBSERVATIONS USED	% Ambiguity Fixed
133	337	UPTC NYSM NYFS	0.014 m	93%	97%
134	752	UPTC NYSM NYFS	0.015 m	92%	98%

    a. The average of the OPUS derived positions was used as the horizontal position and ellipsoidal height of PBM 1-500. The data was downloaded to a PC and processed using the Weighted Ambiguity and Vector Estimator (WAVE) processor in Trimble Geomatics Office, V1.63. The single baseline method was used, with the precise (IGS Rapid) ephemeris. All of the baselines were integer bias fixed solutions. Table H-3 shows the results of the baseline processing:

Table H-3. WAVE Baseline Results.

From	To	UTC Start	Duration Minutes	Length Meters	Ratio	Variance	RMS
MA0095	08042A	5/12/08 19:33	45	32378	21.60	1.5	0.012
MA0095	MA0735	5/12/08 19:33	43	60774	26.48	1.1	0.010
MA0735	08042A	5/12/08 19:24	52	28411	20.69	1.4	0.012
MA0095	08042A	5/13/08 11:30	46	32378	15.82	1.7	0.016
MA0095	MA0735	5/13/08 11:42	34	60774	10.74	0.8	0.013
08042A	MA0592	5/13/08 21:47	30	2435	36.41	12.1	0.011
08042A	MA0592	5/13/08 13:01	30	2435	30.94	2.9	0.005
MA0735	08042A	5/13/08 11:42	46	28411	14.20	1.7	0.015

    b. Each of the baselines was measured twice in independent sessions. The processed vector components were transformed to a local horizon system (north, east, & up) for analysis— see Table H-4.

Table H-4. Baseline Residuals (in meters).

From	To	Delta N	Delta E	Resultant	Delta U	Length
08042A	MA0095	-0.004	-0.003	0.005	-0.001	32379
MA0095	MA0735	0.001	0.008	0.008	0.009	60774
08042A	MA0735	0.001	0.003	0.003	0.013	28412
08042A	MA0592	-0.005	-0.006	0.008	-0.007	2435

H-9. <u>Least Squares Adjustments</u>. The GPS data was adjusted using ADJUST, a least squares adjustment program from the NGS. The processed baselines were parsed to form an input file in the G-file format. The results from the two OPUS solutions were also included. No scaling of the a priori baseline statistics was done. Station errors (HI and centering) of 0.005 m were also included. Geoid separations for each station were interpolated using the GEOID03 model. The first adjustment constrained the CORS UPTC ARP to the published NAD83 (epoch 2002.0) position (latitude, longitude, and ellipsoidal height). The standard deviation of unit weight was

2.42. This value was then used to scale the G file using the "modgee" program. The subsequent adjustment, utilizing the scaled G file, had a standard deviation of unit weight of 1.004. The misclosures at the three NSRS stations and the other two CORS used are shown in Table H-5.

Table H-5. Station Misclosures.

Station	Azimuth	Distance	Δ Ortho H	Δ Ellip H
MA0592 (Z 337)			+0.005 m	
MA0095 (V 25)	217°	0.002 m	-0.004 m	-0.001 m
MA0735 (TTS 64 K)	243°	0.008 m	-0.022 m	-0.008 m
NYSM ARP	285°	0.025 m		+0.006 m
NYFS ARP	280°	0.025 m		-0.002 m

a. The straight line distance between the two HARN bench marks is 60.8 km (37.8 miles), but the distance through the leveling network is about 105 km. Benchmarks Bench marks with that separation could be expected to have a relative accuracy in orthometric height of about 0.01 m between them. The final adjustment constrained PBM 1-500 horizontally and the three existing NSRS bench marks vertically (NAVD88 orthometric height). The estimated variance factor was 1.11. The vertical confidence region at the 95% level for PBM 1-500 from this adjustment was 0.007 m. This, combined with the estimated accuracy of the geoid model, gives an estimated accuracy of the GPS derived orthometric height at PBM 1-500 of ± 0.03 m. An additional check is given by comparing the NAVD88 orthometric height determined in this project to the NAVD88 height determined in 2003 by precise differential levels from **Z 337**, with a difference of 0.004 m.

b. The next adjustment constrained UPTC ARP horizontally and the nearest benchmark to the project, Z 337, vertically (NAVD88 orthometric height). The misclosures in orthometric height at the two HARN/benchmarks were then computed: -0.010 m at V 25 and -0.027 m at TTS 64 K. These misclosures were within the expected range, so the subsequent orthometric height adjustment constrained the three NSRS benchmarks to their published NAVD88 heights, along with the horizontal position of UPTC ARP. The standard deviation of unit weight was 1.34. This adjustment provided the adjusted NAVD88 orthometric height for the new station, EAST BRANCH.

c. The final adjustment constrained the three CORS and the two HARN stations in all three dimensions (latitude, longitude, and ellipsoidal height. The standard deviation of unit weight was 4.42. This adjustment provided the adjusted latitude, longitude, and ellipsoidal height for the new station, EAST BRANCH, as well as the NSRS benchmark Z 337.

H-10. <u>Supplemental Deformation Surveys</u>. A combination of GPS and conventional methods was used in the deformation survey of the six alignment pins nominally online between M1 and M2. A base receiver (Trimble 5700) was running on primary control monument PBM 1-500. Two Trimble R8 GNSS receivers were used to occupy M2, A5, A4, A3, A2, A1, and M1, in order (see Figure H-4). Each occupation had at least 15 minutes common occupation time with adjacent stations. The alignment pins (A1 thru A5) were occupied using a standard tribrach with a precise rotatable optical plummet. Height of antenna measurements were taken as slope measurements to the blue band on the antenna housing, and then corrected to the Antenna

Reference Point (ARP). M1 and M2 were occupied by placing a standard optical plummet directly atop the pedestal. The height of antenna measurements for the pedestals were converted to be the ARP height above the top of the 5/8" bolt. A Trimble S6 high accuracy total station was set up on pedestal M4 located upstream of the dam on the left bank, and angle and distance measurements were taken to each station during the GPS occupations. The distance measurements were corrected for atmospheric conditions and reduced to the mark-to-mark components. The GPS and conventional data were combined in a least squares adjustment to obtain adjusted coordinates for M1, M2, and alignment pins A1 through A5. These coordinates will be directly used in future surveys to monitor the movement of the alignment pins. Offsets of A1 through A5 from the M1→M2 line were also computed to maintain backwards compatibility with previous alignment surveys.
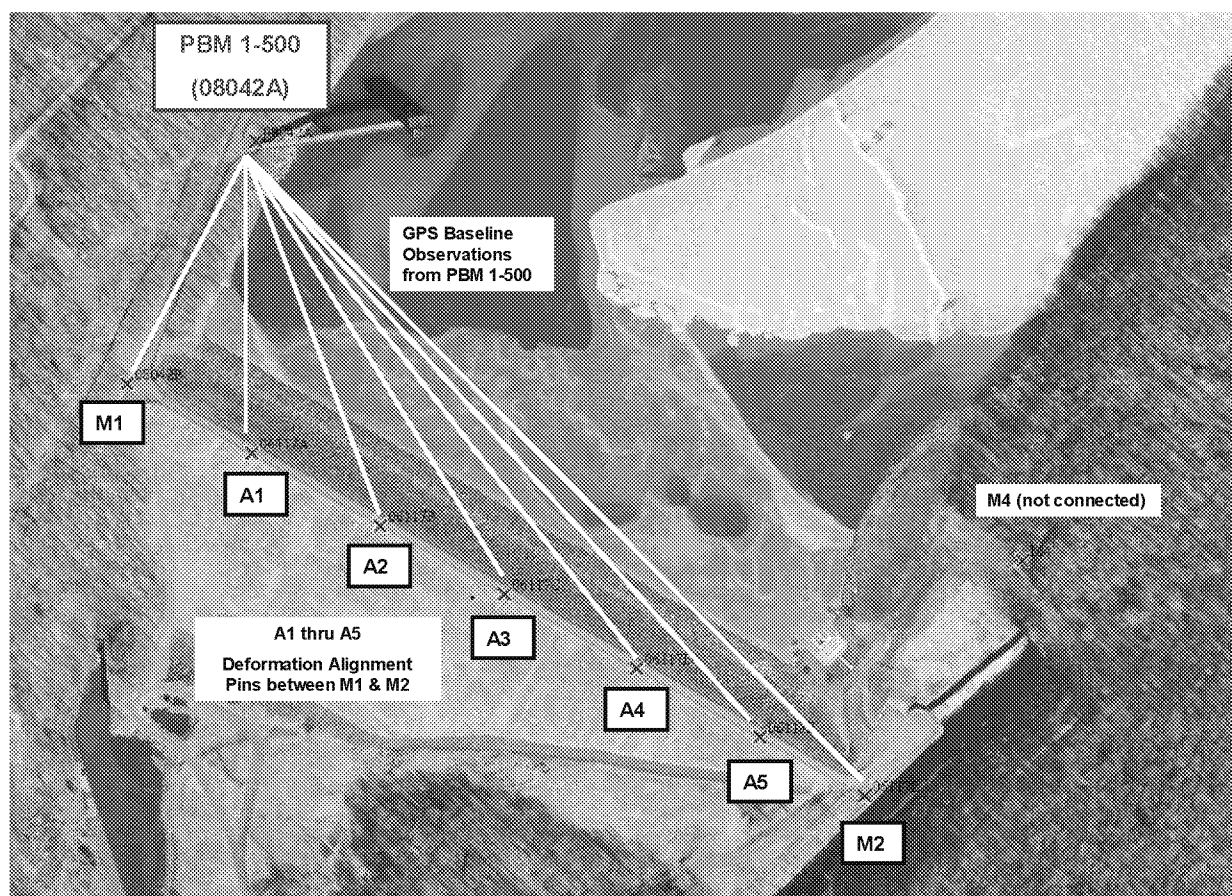

Figure H-4. Local deformation alignment points.

a. Figure H-5 shows the offsets from the line between M1 and M2 over the last several years:
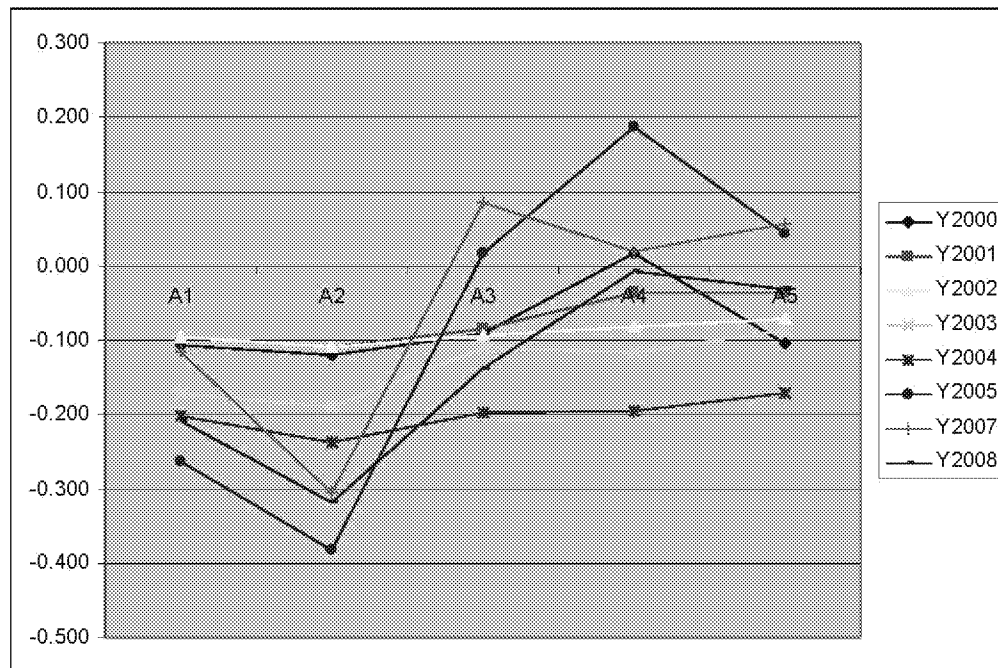

Figure H-5. Alignment point offsets from 2000 to 2008.

b. The settlement survey was executed using a DiNi 12 digital level and bar coded rod. A run was made from PBM 1-500 through each of the pedestals and alignment pins, and back to PBM 1-500 with a loop closure of 0.002 m over a distance of 1.5 km. A spur line was run from A5 down to the outflow area, and continued to the outflow gage located downstream of the dam. A loop was also run from PBM 1-500 to the gage located in the intake tower.

Table H-6. East Branch Clarion River Lake Dam Adjusted Coordinates. PA North Zone State Plane Coordinates – NAD 1983. (NSRS 2007)

Station Name	GPSID	Northing Meters	Easting Meters	NAVD88 Meters	Northing US FT	Easting US FT	NAVD88 US FT	Convergence	Scale Factor	Elevation Factor	Combine Factor
BM 1-500	0804ZA	155221.009	529462.570	521.442	509254.259	1737921.728	1710.784	0°33'33.93"	0.99995998	0.99992305	0.99998304
V 25	MA0995	153851.210	561809.546	239.217	504760.177	1843203.487	1109.834	0°18'16.21"	0.99995956	0.99995185	0.99991140
Z 339	MA8592	154208.738	527347.172	526.969	512563.732	1729810.096	1729.871	0°34'27.87"	0.99996038	0.99992220	0.99998238
TTS 64 K	MA0735	157712.792	801164.981	573.628	517429.386	1664238.776	1881.968	0°47'02.78"	0.99996089	0.99991491	0.99997580
A1	06117A	155026.228	529461.356	520.157	508615.208	1737874.465	1706.548	0°33'33.92"	0.99995991	0.99992325	0.99998316
A2	06117B	154981.284	529541.580	520.182	508467.770	1727337.686	1706.638	0°33'31.63"	0.99995989	0.99992326	0.99998315
A3	06117C	154938.210	529618.493	520.147	508326.484	1737890.005	1706.647	0°33'29.81"	0.99995987	0.99992325	0.99998313
A4	06117D	154891.929	529701.125	520.217	508174.603	1737861.198	1706.748	0°33'27.03"	0.99995986	0.99992325	0.99998311
A5	06117E	154848.789	529778.141	520.313	508033.069	1738113.784	1707.060	0°33'24.82"	0.99995984	0.99992323	0.99998307
M2	06117Z	154812.162	529843.522	521.536	507912.922	1738328.287	1711.073	0°33'22.95"	0.99995982	0.99992304	0.99998286
M1	08042B	155069.880	529383.483	521.504	508758.334	1726818.978	1710.968	0°33'36.18"	0.99995992	0.99992304	0.99998297
M3	M3	154959.204	529941.779	526.281	508395.322	1738650.654	1726.643	0°33'28.19"	0.99995988	0.99992228	0.99998217

   c. Updated NAVD88 elevations at the project site as determined by differential levels
from PBM 1-500 are shown in Table H-7.

Table H-7. Updated NAVD88 Elevations.

Station Name	NAVD88 meters	NAVD88 US FT	Description
1-500	521.442	1710.764	BM on intake bridge parapet wall
A1	520.157	1706.548	Alignment pin
A2	520.182	1706.630	Alignment pin
A3	520.187	1706.647	Alignment pin
A4	520.217	1706.745	Alignment pin
A5	520.313	1707.060	Alignment pin
BOLT	520.385	1707.296	Bolt on floor of intake tower near gage
FLOOR	520.378	1707.273	Floor elevation in intake tower near gage, +0.090 m up to sill
M1	521.504	1710.968	Pedestal (top of bolt)
M2	521.536	1711.073	Pedestal (top of bolt)
M3	521.312	1710.338	Pedestal (top of bolt)
TABLE	521.213	1710.013	Table surface in intake tower, +0.027 m up to knife edge
TBM1	485.854	1594.006	Anchor bolt
TBM2	466.543	1530.650	Square painted on NW corner of building, Weir 6
TBM3	467.505	1533.806	Square painted on south end of left bank training wall at outflow
TBM4	465.819	1528.275	Top of angle iron for weir gage at downstream end of spillway
TBM5	467.489	1533.753	Nail in triple black cherry, upstream of gage house, set by USGS
TBM6	467.331	1533.235	Nail in red maple, downstream of gage house, set by USGS
TBM7	465.774	1528.127	Bolt (lower of 2) protruding from downstream side of gage

   d. A portion of the published NGS NSRS description for PBM 1-500 (i.e., EAST
BRANCH PID = DK7088) is shown below:

```
*******************************************************************
DK7088  HT_MOD       -  This is a Height Modernization Survey Station.
DK7088  DESIGNATION -  EAST BRANCH
DK7088  PID          -  DK7088
DK7088  STATE/COUNTY-  PA/ELK
DK7088  USGS QUAD    -  GLEN HAZEL (1969)
DK7088                      *CURRENT SURVEY CONTROL
DK7088* NAD 83(2007)-  41 33 40.56863(N)     078 35 44.27202(W)     ADJUSTED
DK7088* NAVD 88      -      521.44   (meters)    1710.8   (feet)  GPS OBS
DK7088  EPOCH DATE   -      2002.00
DK7088  X            -    945,126.312 (meters)                     COMP
DK7088  Y            - -4,685,460.249 (meters)                     COMP
DK7088  Z            -  4,209,591.236 (meters)                     COMP
DK7088  LAPLACE CORR-        1.76 (seconds)                     USDV2009
DK7088  ELLIP HEIGHT-      489.730 (meters)        (10/02/08) ADJUSTED
DK7088  GEOID HEIGHT-      -31.70 (meters)                     GEOID09
DK7088  HORZ ORDER  -  B
DK7088  ELLP ORDER  -  THIRD     CLASS II
DK7088.The horizontal coordinates were established by GPS observations
DK7088.and adjusted by the TERRA SURV in October 2008.
DK7088.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
*******************************************************************
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H-11. Comparative Analysis of LIDAR Mapping. A comparative analysis of existing mapping data was performed at the East Branch Dam site in 2009. This involved comparisons with design (as built) data, gage reference elevations, and established pool reference elevations. The comparative analysis performed by Photo Science involved the use of three independent data sources. The "hydraulic" data source used in the analysis consisted of 2006, high-resolution, PaMAP LIDAR elevation data obtained by Photo Science from the Pennsylvania State University, Institute of State and Regional Affairs, Center for Geospatial Information Services located in Middletown, PA. The "geodetic" data source used in the analysis consisted of spillway survey profiles in *.csv format established by Photo Science sub consultant, TerraSurv Inc., in May of 2008. Lastly, the USACE supplied, "East Branch Dam Plan Elevation and Section Drawing" in PDF format dated 30 September 1982 was utilized to compare both the hydraulic and geodetic data sources against the original dam and spillway design elevations. Using these three sources a comparative analysis of the survey profiles along the top of the dam and spillway structures was performed by measuring, comparing, and recording survey elevations along each profile to their respective engineering design elevation and the existing LIDAR surface elevation.

H-12. Data Source Projection/Datum. In order to perform the comparative analysis it was necessary to ensure that all data sources were in the same projection, datum, and units of measurement. The horizontal projection/datum established for the analysis was the Pennsylvania State Plane Coordinate System (PASPCS), North Zone, North American Datum 1983 (NAD83). The vertical datum established for the analysis was expressed in orthometric heights using North American Vertical Datum 1988 (NAVD88). Both horizontal and vertical units were expressed in US Survey Feet.

H-13. East Branch Dam Plan Elevation and Section Drawing. An Adobe PDF file of the East Branch Dam Plan Elevation and Section Drawing was provided by the Pittsburgh District (see Figure H-6). The drawing identifies a design elevation of 1,707.0 feet at the top of the dam structure and 1,685.0 feet at the top of the spillway structure. Although the vertical datum is not explicitly identified on this design drawing, the drawing predates by some 6 years the release of the NAVD88, and therefore an assumption was made for NGVD29 elevations. To support the analysis it was necessary to convert these design elevations to NAVD88. Using NGS BM Z339, which is in the immediate vicinity of the dam and spillway, USACE personnel reviewed the NGS data sheet for BM Z339 and computed a (-) 0.49 foot difference between NGVD29 to NAVD88. Photo Science then applied the (-) 0.49 foot reduction to the NGVD29 design elevations resulting in the computed NAVD88 elevations of 1,706.51 feet for the top of dam structure and 1,684.51 feet for the top of the spillway structure. These computed NAVD88 design elevations were then used in the comparative analysis.
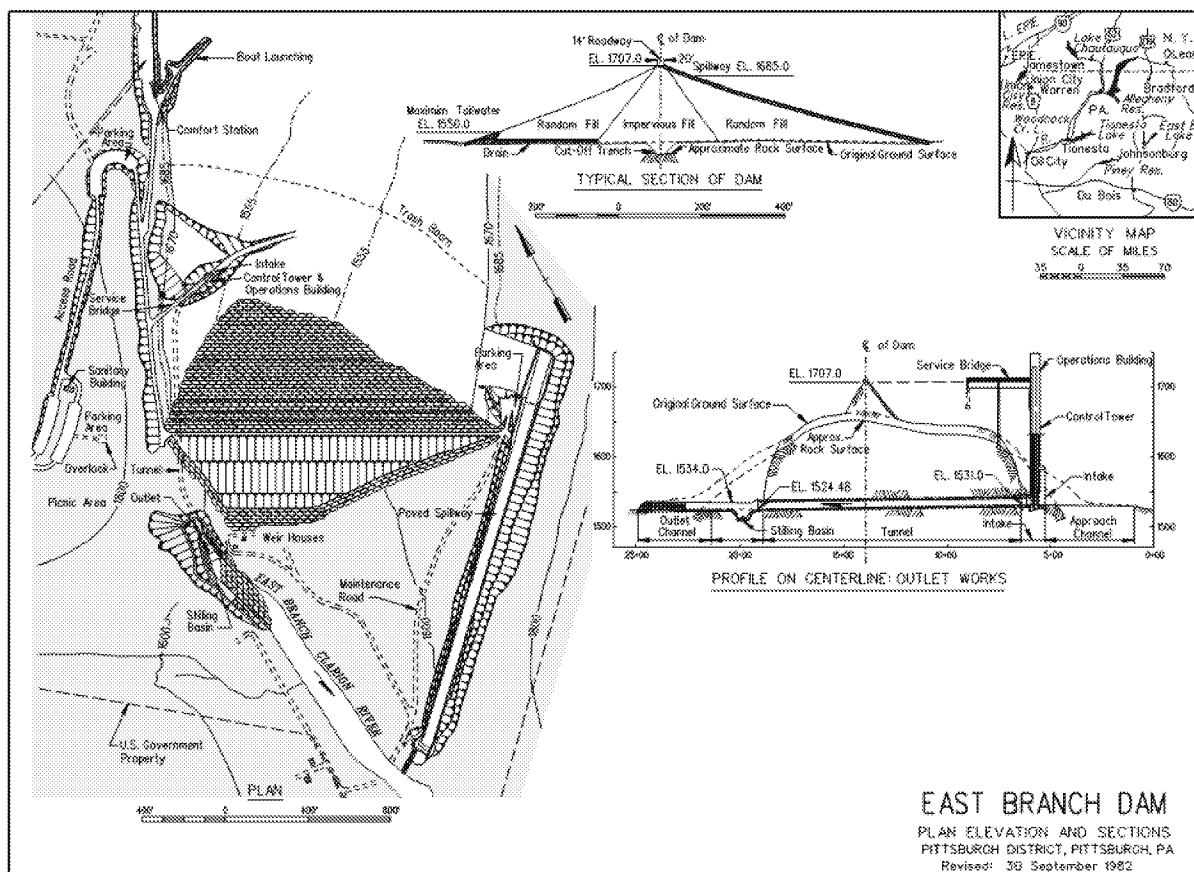
Figure H-6. East Branch Dam plan/elevation drawing.

H-14. <u>Survey Profile Dataset</u>. In May of 2008, Photo Science sub consultant, TerraSurv Inc., performed a series of field surveys at East Brach Clarion River Lake Dam to establish a primary control network, deformation monitoring and profiling. TerraSurv developed profiles along the dam and uncontrolled spillway. The profile data in *.csv format of the dam and spillway obtained under this task order was supplied to Photo Science for use in the comparative analysis. A total of 36 points were collected along the top of the dam and an additional 7 points were collected on the top of the spillway. Figure H-7 depicts the individual profile stations of both the dam and spillway on top of the 2006 PaMAP orthophoto imagery. Photo Science converted the survey data provided by TerraSurv from UTM, Zone 17N, NAD83 (meters) to Pennsylvania State Plane Coordinate System, North Zone, NAD83 (feet). Elevation values were provided in NAVD88, meters and converted to feet. The dam and spillway profile data provided the "geodetic" input in the comparative analysis.

Figure H-7. Dam and Spillway Profile Stations displayed with
2006 PaMAP Ortho Imagery.

H-15. PaMAP LIDAR Elevation Dataset. The "hydraulic" data source used to support the
comparative analysis was the State of Pennsylvania's spring 2006 PaMAP LIDAR elevation
dataset. Photo Science obtained the classified LIDAR point cloud data in native LAS file format
covering the dam and spillway area from the Pennsylvania State University, Institute of State and
Regional Affairs, Center for Geospatial Information Services located in Middletown, PA. The
LIDAR data was acquired by the PaMAP program in the spring of 2006 during leaf off
conditions. As depicted in Figures H-8 and H-9, Photo Science utilized the bare earth point class
contained in the LAS file to create a ground surface of the dam and spillway area.
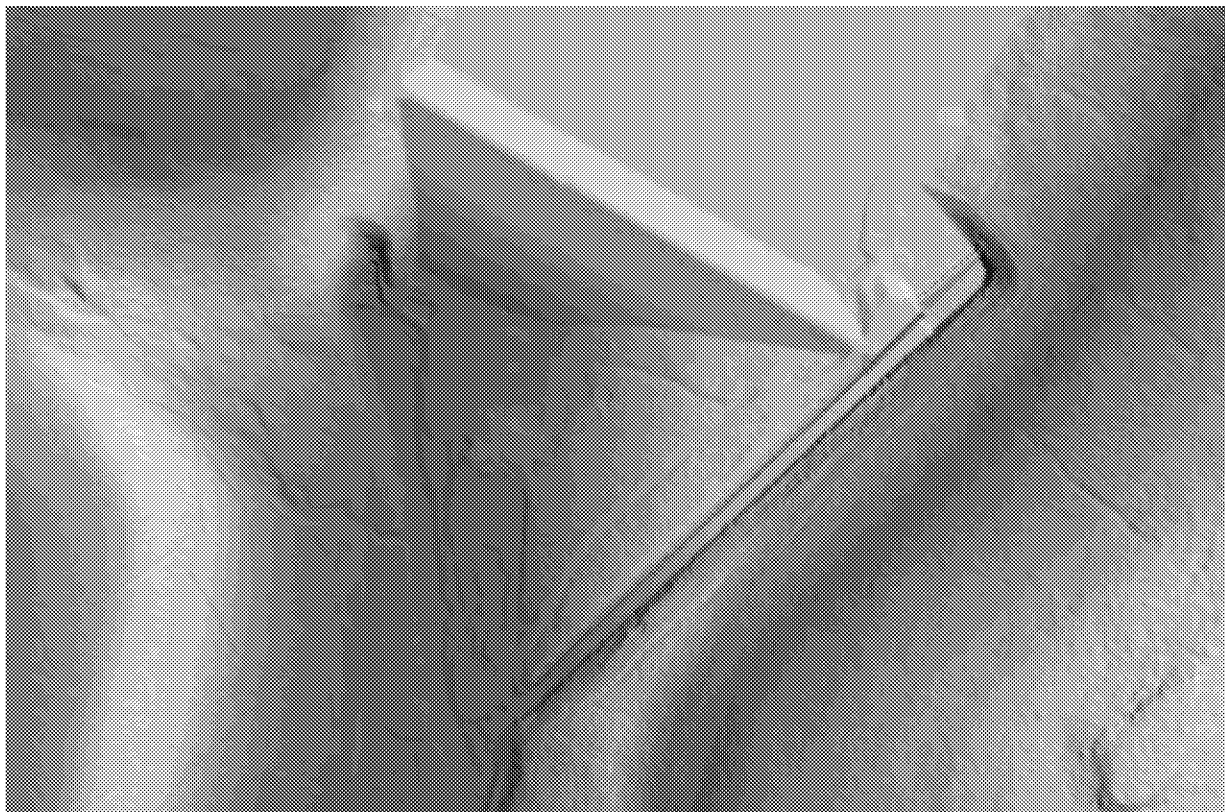
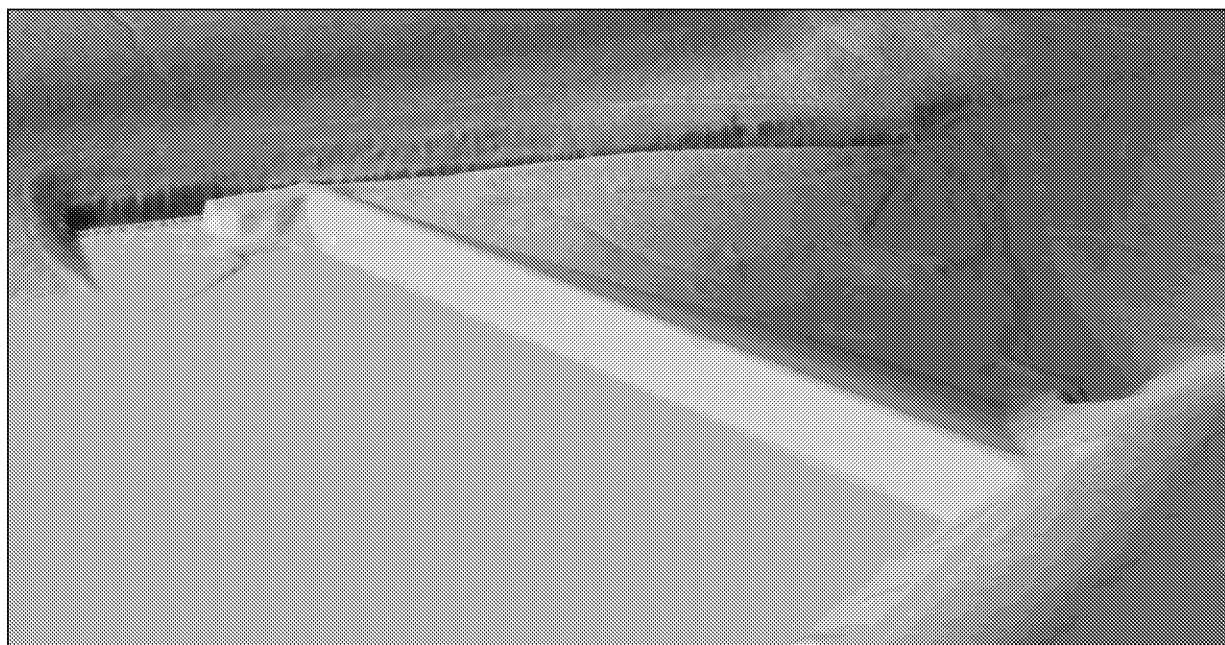Figure H-8. Top View of East Branch Clarion River Dam & Spillway.

Figure H-9. Isometric view above the East Branch Clarion River Dam & Spillway.

a. The PaMAP LIDAR bare earth data set (in Elk County, PA) was designed to achieve 18.5 cm (0.61feet) vertical RMSE for LIDAR bare earth elevation surface in open terrain. The data was independently tested by PaMAP Quality Assurance Consultant, Dewberry. As shown in Figure H-10, the accuracy of the bare earth in open terrain achieved an RMSE of 0.34 feet and a consolidated RMSE of 0.54 feet for all categories tested.

Dewberry

8401 Arlington Boulevard   703 849 0396
Fairfax, Virginia 22031-4666   703 849 0182 fax
www.dewberry.com

Date: April 7, 2008

Subject: Report of Vertical Accuracy Testing of LIDAR Data for LandAirWoolpert Lidar Block

		Counties in Production Block:
Project Name:	PAMAP	• Elk
Product Tested:	LIDAR DTM	• Cameron
State Plane Zone:	Pennsylvania North and South NAD83	• Clearfield
Production Block:	LandAirWoolpert Block, 2006	• Centre
Accepted/Rejected:	Accepted	• Cambria
Production Company:	BAE	• Blair

Table 1. Vertical Accuracy Statistics per NSSDA/FEMA Guidelines

100 % of Totals	RMSE (ft) Open Terr. Spec=0.61ft	Mean (ft)	Median (ft)	Skew	Std Dev (ft)	# of Points	Min (ft)	Max (ft)
Consolidated	0.54	0.22	0.12	2.70	0.49	100	-0.54	2.99
Open Terrain	0.34	0.11	0.12	0.26	0.33	19	-0.40	0.73
High Grass	0.31	0.14	0.11	0.61	0.28	21	-0.26	0.81
Brush	0.74	0.44	0.15	1.53	0.61	20	-0.11	1.99
Forest	0.74	0.34	0.30	3.32	0.68	20	-0.54	2.99
Urban	0.35	0.07	-0.05	1.42	0.36	20	-0.31	0.89

Figure H-10. Dewberry Vertical Accuracy Report of 2006 LIDAR Block covering Elk County, PA.

b. The PaMAP Bare Earth LIDAR surface generated for the dam and spillway provided the "hydraulic" input in the comparative analysis.

H-16. Data Processing. A rectangular polygon was placed around the dam and spillway area, buffered by 1000 feet, for the purposes of reviewing and validating the visual quality the bare earth surface generated from the PaMAP LIDAR data. Minor editing of the bare earth points was performed to improve the quality of the final surface used to perform the analysis. TerraScan and TerraModeler software packages were used to perform for all data classification, manual cleanup, and data analysis. Once the bare earth surface was generated the technician imported the coordinate locations of the survey profiles into the project workspace. These included 36 points along the top of the dam (Figure H-11) and an additional 7 points along the top of the spillway. The profile station locations were then intersected with the 3-D bare earth LIDAR surface. Interpolated LIDAR elevations were then generated using the software at each profile station location. The common coordinates of each survey profile station along with its surveyed elevation, its LIDAR derived elevation and the constant design elevation for the dam and spillway were then exported from TerraScan and imported into Microsoft Excel 2007 for a

final statistical analysis. The tabular and graphed results of the analysis of both the dam and spillway are shown in Section H-20 through H-23.
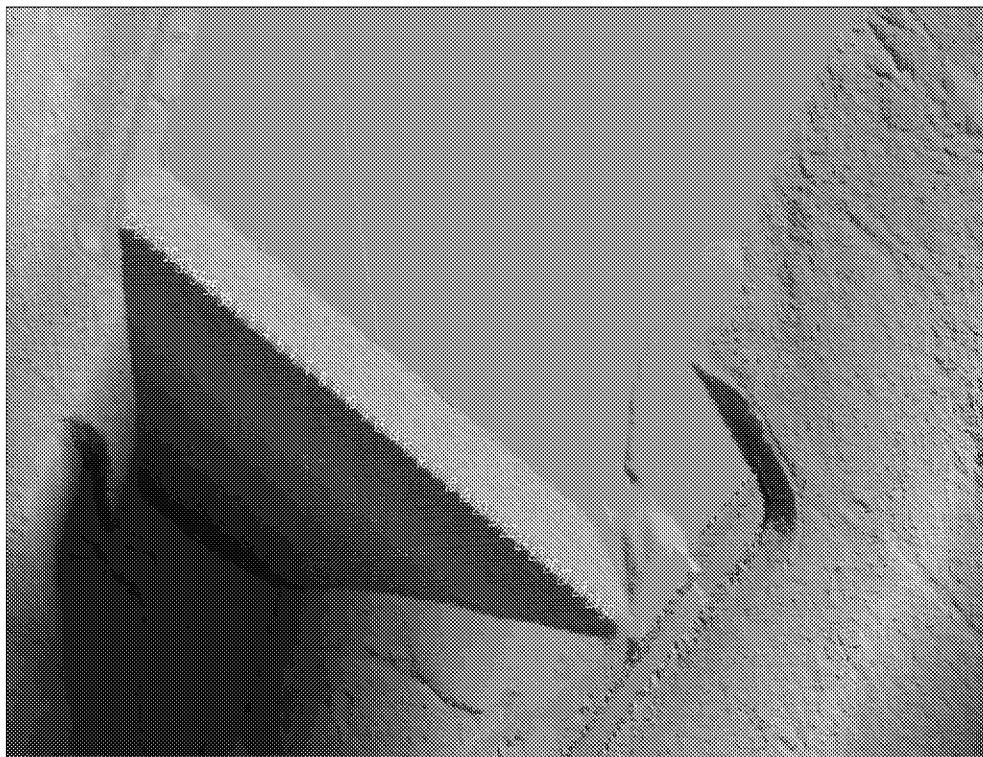

Figure H-11. Bare Earth LIDAR Surface showing Dam profile stations.

H-17. Comparative Analysis Observations. The field survey elevations established along the top of the dam and spillway were consistently higher than the plan elevation. For the dam the magnitude was approximately one half foot and approximately three tenths of a foot for the spillway. The elevations along the top of the dam and spillway established from the LIDAR elevation model were also consistently higher than the plan elevation, but there was considerably more "noise" and variability in the LIDAR elevations as compared to the other two elevation sources. This noise is likely a result of both the nature of LIDAR elevation data, which is acquired in an aerial platform flown several thousand feet above the ground, and the lack of breakline data along the tops of the slopes on the dam that would have improved the performance of the LIDAR only data in modeling the top of the dam. The "noise" in the LIDAR data is not necessarily unusual. As described earlier in this document, this LIDAR dataset was acquired to support a 2-foot contour equivalent surface and as such, included a requirement for an 18.5 cm, or 0.61 feet root mean square error (RMSE). The RMSE basically defines the 68 percent confidence interval, or put in other words, 68 percent of the elevation points within this dataset should fit the actual surface of the earth within 18.5 cm, or 0.61 feet. By visual inspection of the graphs for both the dam and the spillway profiles we can see that the LIDAR elevations fit the ground elevations established by field survey within 0.5 feet at most of the comparison locations, which would fit within our statistical expectations for the LIDAR data based on the accuracy standard

H-18. Project Glossary.

   LIDAR- Light Detection and Ranging.

   Average dZ – the average elevation value from the list of each series of readings.

   Minimum dZ – the minimum elevation value from the list of each series of readings.

   Maximum dZ – the maximum elevation value from the list of each series of readings.

   MSE - Mean Square Error is achieved by calculating the square of the deviations of points from their true position, summing up the measurements, and then dividing by the total number of points.

   RMSE – Root Mean Square Error is calculated by taking the square root of the MSE.

   Standard Deviation – measure of how widely values are dispersed from the average dZ.

H-19. Methodology for Calculating the dZ Values.

   dZ (Survey/Plan) – Plan elevation was subtracted from the surveyed elevation.

   dZ (Survey/LIDAR) – LIDAR elevation was subtracted from the survey elevation.

   dZ (Plan/LIDAR) – LIDAR elevation was subtracted from the plan elevation.

H-20. Statistical Analysis—Dam.

Dam Statistical Analysis			
Average Dz	0.56	-0.278	-0.838
Minimum Dz	0.39	-0.70	-1.33
Maximum Dz	0.87	0.28	-0.22
RMSE	0.571	0.398	0.899
St. Dev.	0.11	0.289	0.33

Point	Elevation			dZ			Easting	Northing
	Surveyed	Plan	LIDAR	Survey/Plan	Survey/LIDAR	Plan/LIDAR		
D_1	1707.38	1706.51	1707.44	0.87	-0.06	-0.93	1736840.37	508754.83
D_2	1706.94	1706.51	1707.14	0.43	-0.20	-0.63	1737189.90	508558.95
D_3	1706.98	1706.51	1706.93	0.46	0.04	-0.42	1737229.23	508537.05
D_4	1706.90	1706.51	1706.75	0.39	0.15	-0.24	1737269.25	508514.86
D_5	1706.94	1706.51	1706.92	0.43	0.02	-0.41	1737309.73	508492.06
D_6	1707.00	1706.51	1706.73	0.49	0.27	-0.22	1737349.08	508469.89
D_7	1707.01	1706.51	1707.52	0.50	-0.51	-1.01	1737390.15	508446.86
D_8	1706.99	1706.51	1707.47	0.47	-0.49	-0.96	1737429.28	508425.16
D_9	1706.96	1706.51	1707.53	0.44	-0.58	-1.02	1737468.98	508402.62
D_10	1706.98	1706.51	1707.45	0.47	-0.47	-0.94	1737508.45	508380.57
D_11	1707.00	1706.51	1707.62	0.49	0.62	1.11	1737549.04	508357.56
D_12	1707.06	1706.51	1707.16	0.55	-0.10	-0.65	1736879.94	508733.03
D_13	1707.03	1706.51	1707.43	0.52	-0.40	-0.92	1737590.29	508334.64
D_14	1707.06	1706.51	1707.59	0.55	-0.53	-1.08	1737630.73	508312.62
D_15	1707.10	1706.51	1707.60	0.59	-0.50	-1.09	1737671.51	508289.43
D_16	1707.05	1706.51	1707.30	0.54	-0.25	-0.79	1737711.33	508267.09
D_17	1707.03	1706.51	1707.73	0.52	-0.70	-1.22	1737752.00	508244.48
D_18	1707.02	1706.51	1707.33	0.51	-0.31	-0.82	1737792.72	508221.78
D_19	1707.05	1706.51	1707.51	0.54	-0.46	-1.00	1737832.91	508199.68
D_20	1707.10	1706.51	1707.62	0.59	-0.52	-1.11	1737873.90	508176.72
D_21	1707.11	1706.51	1707.59	0.60	-0.48	-1.08	1737914.50	508154.14
D_22	1707.10	1706.51	1707.62	0.59	-0.52	-1.11	1737953.58	508131.90
D_23	1707.13	1706.51	1707.09	0.62	0.04	-0.58	1736918.46	508711.93
D_24	1707.09	1706.51	1707.61	0.58	-0.52	-1.10	1737993.20	508109.57
D_25	1707.19	1706.51	1707.59	0.68	-0.40	-1.08	1738034.14	508086.55
D_26	1707.25	1706.51	1707.84	0.74	-0.59	-1.33	1738074.32	508063.43
D_27	1707.34	1706.51	1707.68	0.83	-0.34	-1.17	1738114.04	508041.69
D_28	1707.31	1706.51	1707.76	0.80	-0.45	-1.25	1738155.30	508018.97
D_29	1707.16	1706.51	1707.70	0.65	-0.54	-1.19	1738193.35	507997.10
D_30	1707.06	1706.51	1707.60	0.55	-0.54	-1.09	1738232.99	507974.79
D_31	1707.09	1706.51	1707.07	0.58	0.02	-0.56	1736957.60	508690.05
D_32	1707.02	1706.51	1706.99	0.51	0.03	-0.48	1736995.53	508667.85
D_33	1707.05	1706.51	1706.77	0.54	0.28	-0.26	1737036.58	508644.95
D_34	1707.07	1706.51	1707.10	0.56	-0.03	-0.59	1737074.70	508623.45
D_35	1707.01	1706.51	1706.89	0.50	0.12	-0.38	1737111.47	508602.91
D_36	1706.98	1706.51	1706.86	0.47	0.12	-0.35	1737151.46	508580.47

H-19

H-21. <u>Dam Statistical Analysis – Graph</u>.
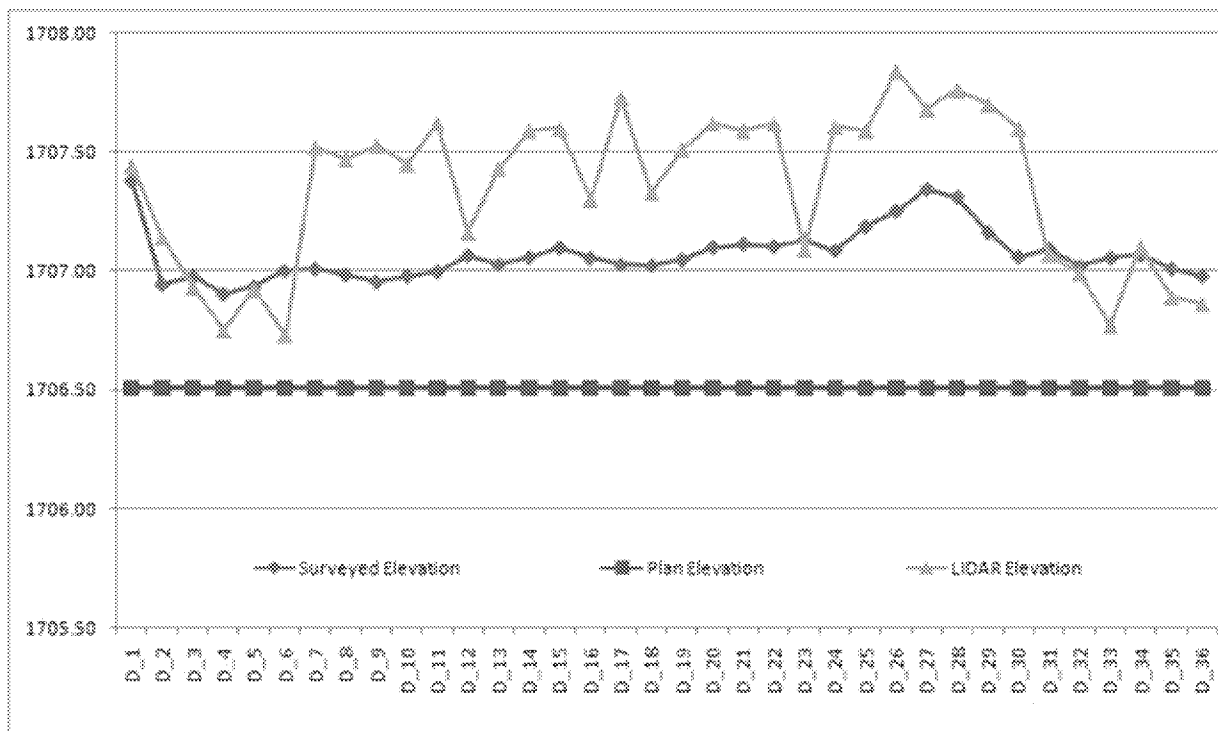
H-22. <u>Statistical Analysis—Spillway</u>.

		Spillway Statistical Analysis				
		Average Dz	0.256	-0.024	-0.28	
		Minimum Dz	0.23	-0.48	-0.75	
		Maximum Dz	0.28	0.41	0.13	
		RMSE	0.256	0.272	0.387	
		St. Dev.	0.019	0.293	0.289	

Point	Elevation			dZ			Easting	Northing
	Surveyed	Plan	LIDAR	Survey/Plan	Survey/LIDAR	Plan/LIDAR		
S_31	1684.78	1684.51	1685.26	0.27	-0.48	-0.75	1738491.30	508110.83
S_32	1684.76	1684.51	1684.82	0.25	-0.06	-0.31	1738513.26	508128.62
S_33	1684.75	1684.51	1684.69	0.24	0.06	-0.18	1738535.40	508146.52
S_34	1684.78	1684.51	1684.77	0.27	0.01	-0.26	1738557.30	508164.61
S_35	1684.79	1684.51	1684.38	0.28	0.41	0.13	1738579.46	508182.41
S_36	1684.75	1684.51	1684.58	0.24	0.17	-0.07	1738601.59	508200.15
S_37	1684.74	1684.51	1685.03	0.23	-0.29	-0.52	1738624.87	508219.20

H-23. <u>Spillway Statistical Analysis – Graph</u>.
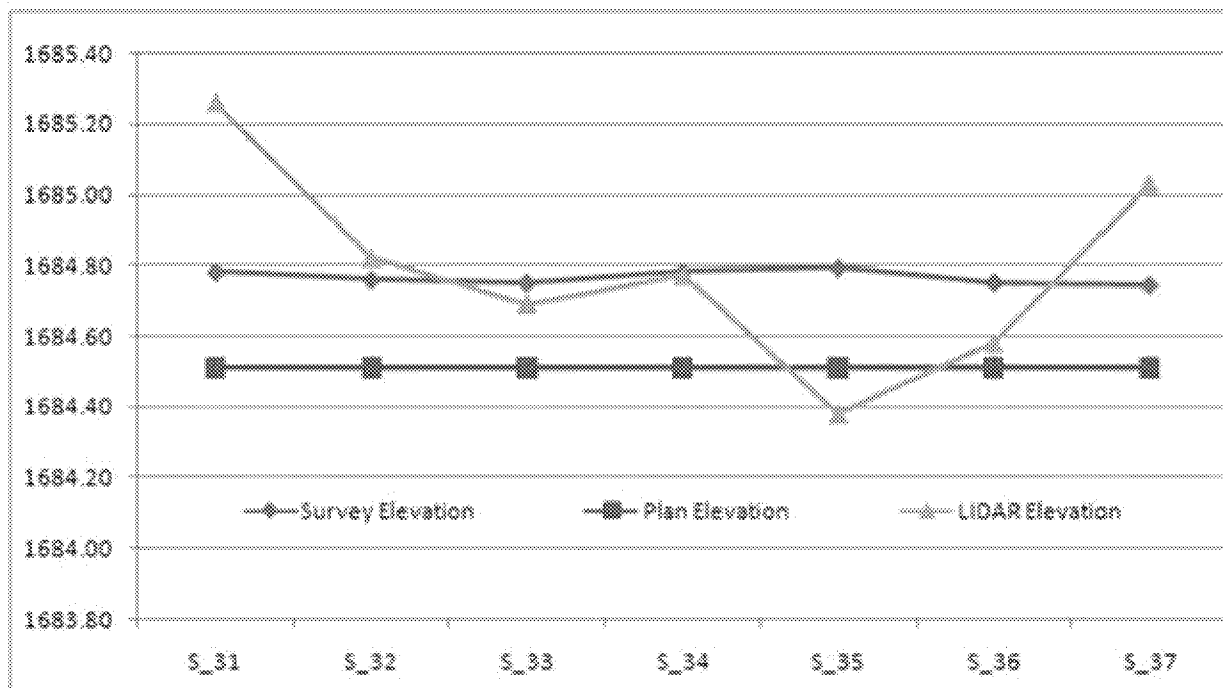
H-24. East Branch Control Tower Gage. The following is a copy of a 2008 USGS gage inspection report for the Control Tower gage. Note that elevations are referenced to both "MSL" and local gage (and electric tape) datums. The 2008 TerraSurv surveys described above subsequently provided relationships for this gage to the NSRS (NAVD88). The legacy datum (MSL) should be retained along with its relationship to the updated NAVD88 elevations. Figure H-12 is a close up of the gage reference point. Figure H-13 is a copy of the U-SMART datasheet for this gage.

Gage Station Description 03027000
East Branch Clarion River Lake, PA

Responsible Office U.S. Geological Survey, Pittsburgh Field Office, 1000 Church Hill Rd., Pittsburgh, PA 15205 (412) 490-3800

Most recent revision: 5/13/2008 Revised by: ajruddy

LOCATION.--Lat 41°33'35", long 78°35'40" referenced to North American Datum of 1927, Elk County, PA, Hydrologic Unit 05010005, gage house in control tower at East Branch Clarion River Dam on East Branch Clarion River, 1.7 miles northeast of Glen Hazel, and 7.5 miles upstream from confluence with West Branch Clarion River.

ROAD LOG.--To reach station from Johnsonburg travel east on Bendingo Rd. from Johnsonburg to village of Glen Hazel. At Glen Hazel make left turn at "T" intersection onto Glen Hazel Rd. (SR240011). Follow Glen Hazel Rd. across bridge over the East Branch Clarion River and proceed 1.0 mile. Make right turn onto Corps of Engineers access road at sign. Follow access road to Corps office and obtain key for access and directions if necessary (Glen Hazel, 7 1/2 minute quadrangle).

DRAINAGE AREA.--72.4 mi2.

ESTABLISHMENT AND HISTORY.--June 1952 to Oct. 1991 and from July 2005 to current year. Prior to October 1970 published as "East Branch Clarion River Reservoir".

GAGE.--Sutron (model 8210) data collection platform at top of concrete stilling well built into gate tower building. Recorders are referenced to an electric tape-gage at 1710.323 ft. gage datum. Elevation of gage datum provided by Corps of Engineers. Electric tape index is 210.323 ft gage datum. Electric tape-gage reading plus 1,500.00 equals reservoir elevation to sea level. Corps Conventions for Recording Pool Levels: E.T. plus 1500 ft is sea level for pool reading. Electric Tape reading minus 85 ft is DCP reading for transmissions.

RESERVOIR: Rock faced earthfill dam with a capacity of 83,300 acre-ft. Range in usual operation is between 1,651 ft and 1,670 ft above sea level. Full range of operation is between 1,555 ft (sill of outlet gates) and 1,685 ft (full pool).

CONTROL.--Spillway and gate opening are the control factors.

DISCHARGE.--Controlled by two outlet gates whose dimensions are 3' by 4' and 1' by 1.5'.

FLOODS.--The high water of June 24, 1972 reached an elevation of 1,685.55 ft (85,010 acre-ft).

POINT OF ZERO FLOW.--Elevation of sill of the outlet gates is 15.0 ft. gage datum.

REGULATION AND DIVERSIONS.--Reservoir is operated for flood control, low-flow augmentation of Clarion River and recreational use.

ACCURACY.--Records good.

COOPERATION.--The station is maintained cooperatively by the U.S. Army Corps of Engineers and the U.S. Geological Survey.

REFERENCE MARKS.-- BM 1-500 COE brass disk on top of parapet wall at service bridge to control tower on upstream right bank. Elevation is 1711.003 ft, MSL. BM M-1 COE pipe monument right bank just downstream of dam access road. Elevation is 1711.209 ft, MSL. BM M-2 COE pipe monument, left bank just downstream of dam access road. Elevation is 1711.293 ft, MSL.

DATE OF LAST LEVELS. Last run: Jun 15, 2006; Next run: Jun 14, 2009; Frequency: 3 years; Status: OPEN

Figure H-12. East Branch Control Tower gage reference point.
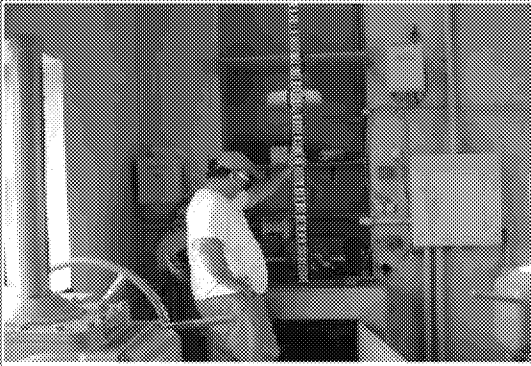
USACE Survey Marker Archive & Retrieval Tool Datasheet   Type: New

Designation: Gauge East Branch (inflow)

Project: East Branch

Stamping:

PID NGS: COE:

State: Pennsylvania

County: Elk

District Pittsburgh

Nearest Town: Johnsonburg

USGS Quad:

T.R.S.:

Nearest Hwy/Mi:

B/L Sta/Off:

Date Recovered: Apr 22, 2008

By: CELRP (LeBlanc)

Condition/Stability: Good

Setting/Monument Type:

Owner:

GPS Suitable: ◯ Yes   ⦿ No

Obstructions: ☐ N ☐ E ☐ S ☐ W

- Horizontal -
Datum: NAD83   (   )
Lat: 41.36140000   N
Lon: 78.39470000   W
Local Accuracy: 10-m+
NSRS Accuracy: 10-m+
   Survey/Computation Method:
Scaled
Date Observed: May 20, 2008

- Vertical -
Datum: NAVD88   (   )
Elevation Ht: 1,710.323
Ellip Ht: Ft
Local Accuracy: 2-cm
NSRS Accuracy: 0.25'
   Survey/Computation Method:
Geodetic Levels
Date Observed: May 20, 2008   Geoid09

Description/Comments:
East Branch Clarion River Lake, PA (EBRP)

Rovergages.com
http://www2.mvr.usace.army.mil/WaterControl/stationinfo2.cfm?sid=EBRP1&fid=EBRP1&dt=S

Access:

- Tidal/Hydraulic Gauge Relationships -
Owner/Code: USGS
Gauge ID: EBRP   Epoch:

- Datum -   **- Elevation -**
Gage Index   210.323
DCP Datum   85
Select
Select

Zone: Northing: Easting: Convergence: CSF:

- Horizontal View -   **- Close-Up View -**

Required Fields in Red   [Reset Form] [Submit]   U-SMART ver 1.1
                                                               9/24/2009

Figure H-13. U-SMART datasheet for East Branch Control Tower gage.

APPENDIX I

Control Surveys: Bois Brule Levee and Drainage District (St. Louis District)

I-1. Purpose. This appendix provides an example of establishing primary NSRS control and supplemental local control on a levee segment along the Mississippi River. This example is typical of the survey procedures employed to establish NSRS control on any levee segment. This project was completed as part of the St. Louis District's effort in updating levee inventory information for inclusion in the National Levee Database (NLD). The database survey was performed by PBS&J—reference report "National Levee Foot Print Database Surveys," Contract W9133L-05-D-0003 DJ06, dated 19 May 2008.

I-2. Project Location. The Bois Brule Levee and Drainage District is located in northern Perry County, Missouri. The protected area is located on the right bank of the Mississippi River. The total length of flood protection is 38.84 miles long. This includes 38.7 miles of earthen levee (204,308.82 feet), 0.03 miles (146.52 feet) of floodwall, and 0.04 miles (190.99 feet) of closure structures. The protected area is roughly 26,350 acres.

I-3. Survey Control Methods Used to Connect Levees to the NSRS. GPS (RTK) survey methods were employed to establish control on various levee segments along the Mississippi River, as shown in Figure I-1. In the St. Louis area, a RTN network was used. North and south of the St. Louis RTN coverage (including the Bois Brule Levee District), standard RTK methods were employed. This involved recovering at least two published NSRS control points near the levee segment and using these points as a RTK base station. RTK checks between NSRS points were made to confirm the reliability of the NSRS points. Supplemental topographic surveys of levee features were made using RTK techniques.
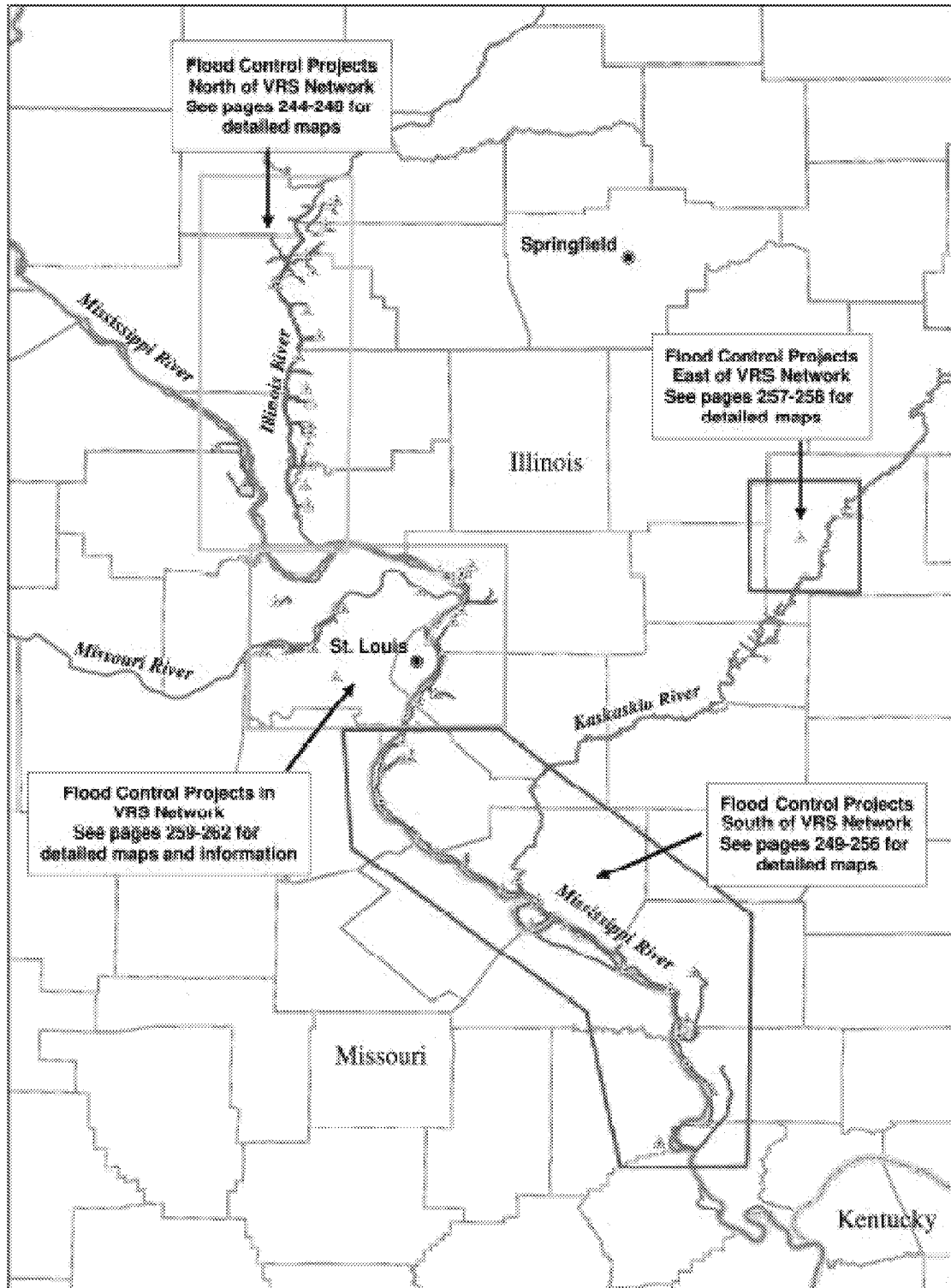
Figure I-1. Overview of survey control used to reference St. Louis District levees.

I-4. <u>Bois Brule Primary Control Points</u>. As shown on Figure I-2, two NSRS control bench marks were recovered in the vicinity of the Bios Brule Levee District—"R 323" (PID=HB1394) and "L 289" (PID=HB1377). These two NSRS points are approximately 10 miles apart. They were designated as PPCPs for this levee project. They are close enough to check internal RTK site calibration. Both points can be occupied with RTK base stations. NGS datasheets for these two points are at the end of this appendix.
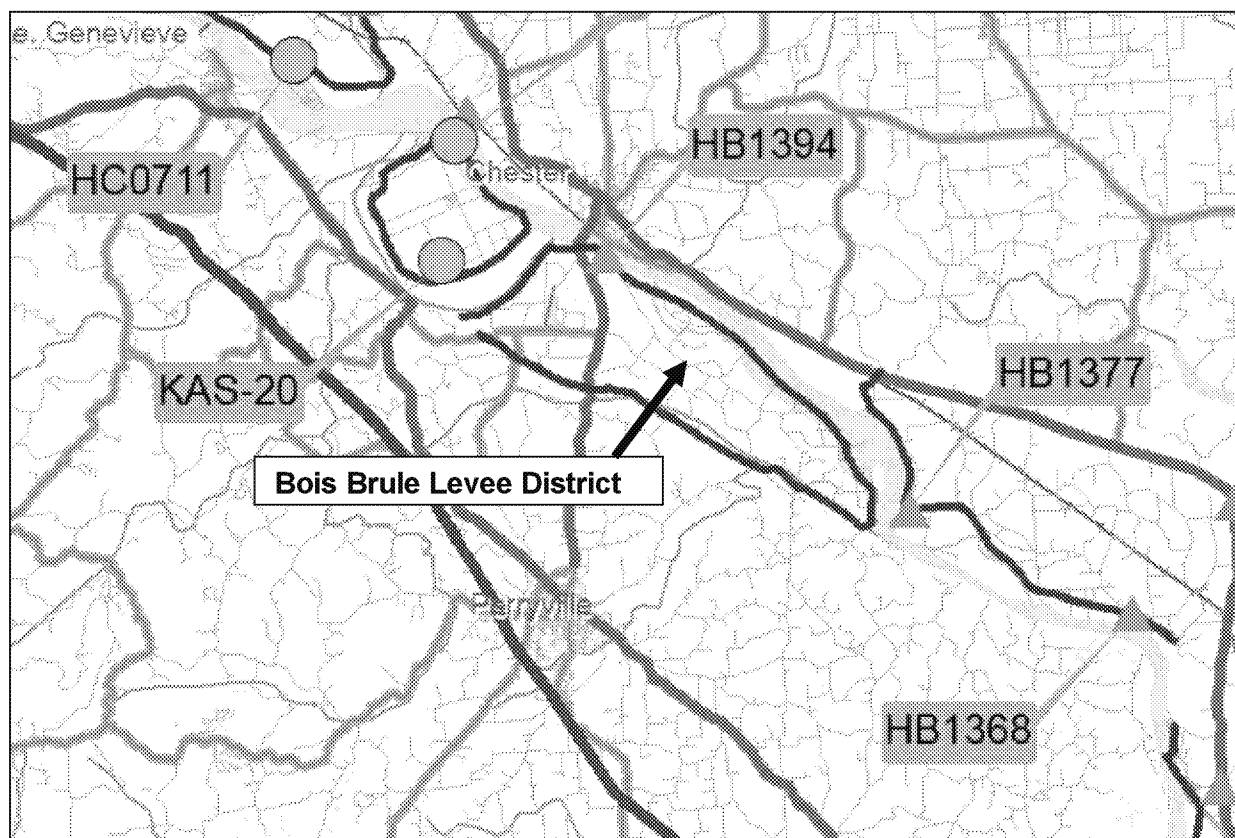
Figure I-2. NSRS control recovered vicinity of Bois Brule Levee District.

I-5. <u>Survey Control / Data Collection</u>. The following Figures I-3, I-4, and I-5 depict the primary NSRS control relative to the levee district boundary and the updated field station descriptions prepared for the NLD inventory report.
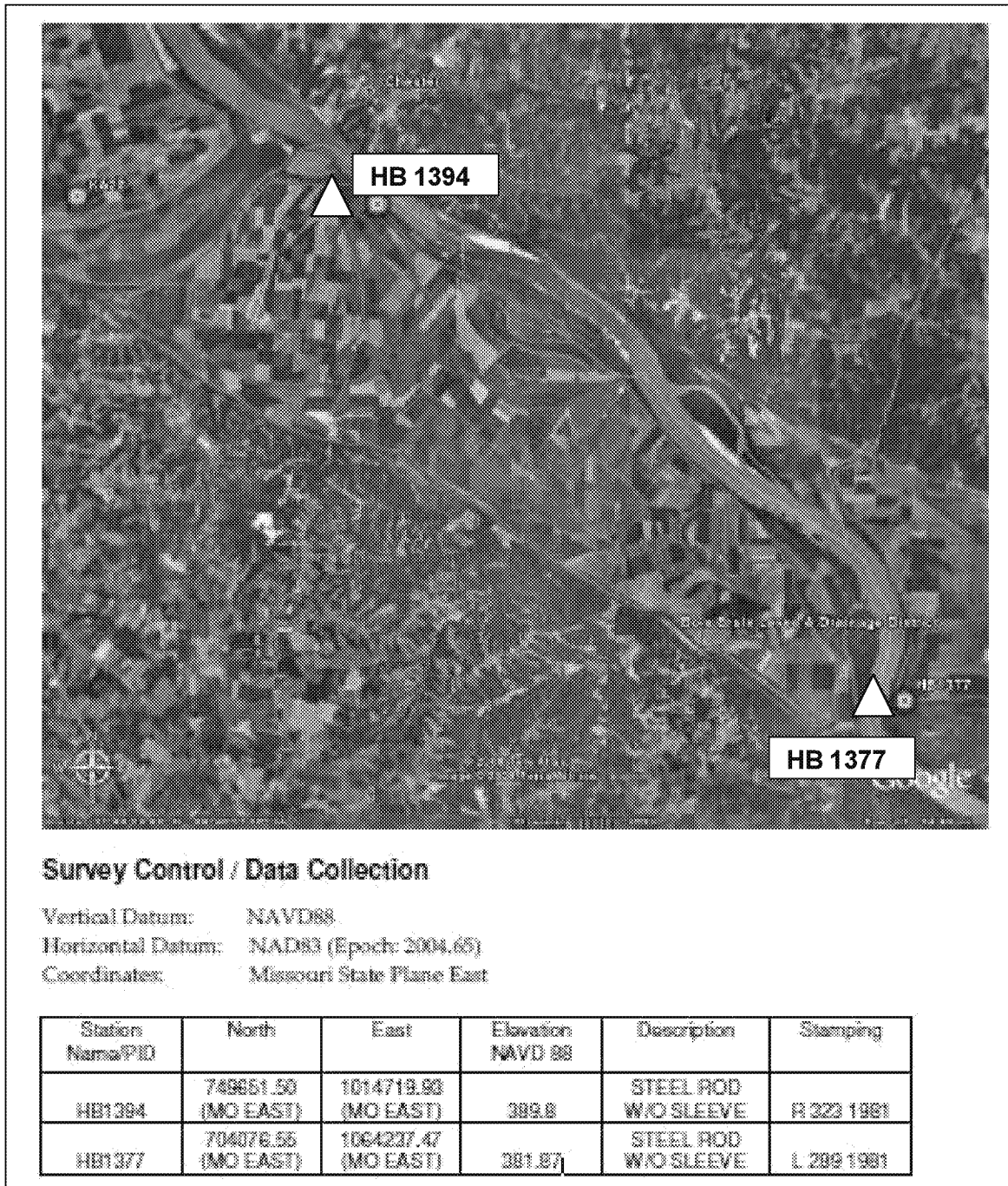

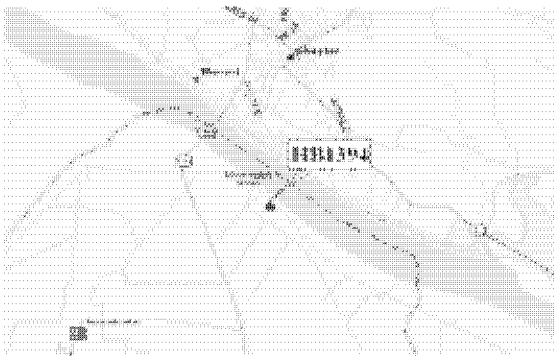Figure I-3. NSRS control for PPCPs HB1394 and HB1377 (Bois Brule Levee District).
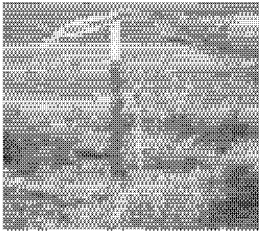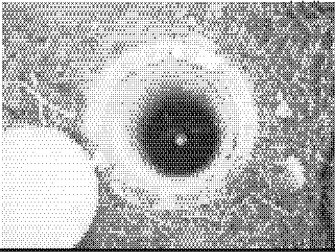
Figure I-4. Datasheet description for PBM "R 323" (HB1394) (Bois Brule Levee District).

St. Louis District National Levee Inventory Database

Survey Control Data Sheet

District: St. Louis

Station: L 289
NGS PID: HB1377
Vicinity Map: DeLorme Street Atlas USA 2006
State: IL
Region: Mid West
Nearest Town: Jacob, IL
USGS Quad: ROCKWOOD (1994)
Nearest Hwy: I-55
Hwy Mile Post: N/A
B/L Station: N/A
B/L Offset: N/A
Levee: Bois Brule, Degognia

Station Name: L 289	NAD83	Controlling Station:
NGS PID: HB1377 Date Set: 1981 Set By: NGS Date Recovered: N/A Recovered By: N/A Owner: NGS	Lat: 37 45 51.30645 Lon: 89 39 21.89599 MO SPCS83 East Zone US SF Northing: 704076.55 Easting: 1064237.47' NAVD88/2004.65 Elev: 351.97' Scale Factor/Convergence: N/A	N/A Survey Computation Method: The orthometric height was determined by differential leveling and adjusted by the NATIONAL GEODETIC SURVEY in June 1991. Horizontal was determined using static, dual-frequency observation. Local Network Accuracy: N/A N/A

Monument Location: 8.1 KM (5.03 MI) SOUTH FROM CORA. 8.1 KILOMETERS (5.05 MILES) SOUTH ALONG THE TOP OF THE MAIN LEVEE FROM THE INTERSECTION OF THE MISSOURI PACIFIC RAILROAD IN CORA TO A BEND IN THE LEVEE AND THE MARK ON THE LEFT, AT THE JUNCTION OF A SPUR LEVEE LEADING SOUTHEAST, IN LINE WITH THE CENTER OF THE SPUR LEVEE, 4.57 METERS (15.0 FEET) NORTH OF THE CENTER OF THE LEVEE ROAD AND 0.46 METERS (1.5 FEET) EAST OF A METAL WITNESS POST. THE MARK IS 0.46 METERS W FROM A WITNESS POST. THE MARK IS 0.15 M BELOW TOP OF LEVEE.

Monument Description: STAINLESS STEEL ROD IN SLEEVE (10 FT +)

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Figure I-5. Datasheet description for PBM "L 289" (HB1377) (Bois Brule Levee District).
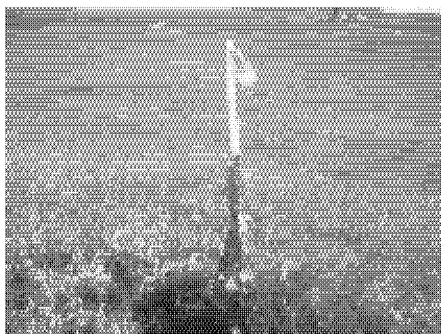
I-6. <u>Bois Brule Levee District Project Features Surveyed Relative to NSRS Primary Control Points HB1394 and HB1377</u>. RTK feature and topographic surveys were performed relative to the PPCPs cited above. The top elevation of the levee varies from 381.94 to 395.09 feet with an average top width of 7.43 feet along the earthen levee sections. The features associated with the levee structure are listed below:

> *Boreholes: None Captured*
> *Encroachment Points: 183*
> *Flood Fight Points: None Captured*
> *Crossing Points: 141*
> *Failure Points: None Captured*
> *Relief Wells: 427*
> *Piezometers: 23*
> *Pump Stations: 4*
> *Sand Boils: None Captured*
> *Closure Structure Count: 3*
> *Cross Sections: 29*
> *Floodwall Lines: 4*
> *Gravity Drains: 26*
> *Rehab Lines: None Captured*
> *Toe Drains: None Captured*

The twenty-nine cross-sections were taken along the levee at stations 8+07, 75+34, 150+98, 182+25, 237+31, 313+64, 371+80, 436+29, 493+32, 575+88, 637+63, 704+83, 790+03, 832+25, 880+66, 920+53, 971+90, 1037+63, 1103+81, 1144+14, 1208+87, 1272+00, 1336+40, 1388+08, 1476+63, 1541+14, 1581+70, 1640+48, and 1725+92.

I-7. <u>Use of RTN Networks for Referencing Levee Control</u>. Figure I-6 depicts RTN network coverage in the St. Louis region. Levee control and NLD inventory points can be directly surveyed from such an RTN network. Prior to performing supplemental surveys, the RTN network is calibrated against existing NSRS control in the vicinity of the levee—as highlighted in green in Figure I-6. Figures I-7 and I-8 show the results of the RTN calibration checks at selected NSRS points. These results indicate vertical checks are within ± 0.25 ft tolerances. The calibration differences at NSRS "tie points" should be applied to local observations—i.e., "site calibration."

Figure I-6. RTN coverage in the St. Louis region and NSRS check points.

VRS BASE STATION	NGS STATION	DISTRICTS USED IN
SRDX	DI2212	Monarch Chesterfield
SIAI	(No PID)	Metro East, Chain of Rocks, Wood River
SIHQ	DH7921	Metro East, Chain or Rocks, Prairie DuPont, Columbia
WIFH	DI2210	Metro East, Chain of RocksPrairie DuPont
TWMW	DI2208	Prairie DuPont, Columbia, Fish Lake
FWIF	(No PID)	Nutwood, Eldred,Spankey

Figure I-7. RTN (VRS) site calibration points for various levee segments in St. Louis area.

RMS TABLE

PROJECT:	Midwest RTK Network RMS Check
DATE:	April 30, 2007

TIE POINT	N	E	Z	REF BM	N	E	Z
AA8597	1083358.929	777759.266	528.897	AA8597	1083358.936	777759.200	529.0
AA8631	1076448.529	833046.498	458.900	AA8631	1076448.429	833046.410	459
AA8649	1094248.925	894738.346	486.682	AA8649	1094248.849	894738.276	487
AA8723	965171.227	882610.836	416.358	AA8723	965171.290	882610.656	416.4
JC0051			527.494	JC0051			527.73
JC1107			413.786	JC1107			413.99
JC1132			426.435	JC1132			426.32
JC1212 (O/S)			468.830	JC1212 (O/S)			468.93
JC1225 (O/S)			554.390	JC1225 (O/S)			554.52

HORIZ.RMS =	0.13
VERT.RMS =	0.17

Figure I-8. RTN (VRS) site calibration "Published – Observed" differences.

I-9

I-8. NGS Data Sheet for R 323 (PID HB1394).

```
HB1394 ********************************************************************
HB1394  FBN         -   This is a Federal Base Network Control Station.
HB1394  DESIGNATION -   R 323
HB1394  PID         -   HB1394
HB1394  STATE/COUNTY-   MO/PERRY
HB1394  USGS QUAD   -   CHESTER (1993)
HB1394
HB1394                        *CURRENT SURVEY CONTROL
HB1394  _____
HB1394* NAD 83(2007)-  37 53 25.83293(N)    089 49 33.44072(W)     ADJUSTED
HB1394* NAVD 88     -       118.811  (meters)    389.80   (feet)   ADJUSTED
HB1394  _____
HB1394  EPOCH DATE  -       2002.00
HB1394  X           -      15,309.625 (meters)                       COMP
HB1394  Y           -  -5,039,949.530 (meters)                       COMP
HB1394  Z           -   3,895,914.999 (meters)                       COMP
HB1394  LAPLACE CORR-          0.93  (seconds)                     USDV2009
HB1394  ELLIP HEIGHT-         89.321 (meters)          (02/10/07) ADJUSTED
HB1394  GEOID HEIGHT-        -29.48  (meters)                      GEOID09
HB1394  DYNAMIC HT  -        118.730 (meters)     389.53  (feet)    COMP
HB1394
HB1394  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
HB1394  Type     PID   Designation                  North   East  Ellip
HB1394  -----------------------------------------------------------------
HB1394  NETWORK HB1394 R 323                         0.41   0.31   1.12
HB1394  -----------------------------------------------------------------
HB1394  MODELED GRAV-    979,943.3   (mgal)                       NAVD 88
HB1394
HB1394  VERT ORDER  -  FIRST     CLASS II
HB1394
HB1394.The horizontal coordinates were established by GPS observations
HB1394.and adjusted by the National Geodetic Survey in February 2007.
HB1394
HB1394.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
HB1394.See National Readjustment for more information.
HB1394.The horizontal coordinates are valid at the epoch date displayed above.
HB1394.The epoch date for horizontal control is a decimal equivalence
HB1394.of Year/Month/Day.
HB1394
HB1394.The orthometric height was determined by differential leveling
HB1394.and adjusted in June 1991.
HB1394
HB1394.The X, Y, and Z were computed from the position and the ellipsoidal ht.
HB1394
HB1394.The Laplace correction was computed from USDV2009 derived deflections.
HB1394
HB1394.The ellipsoidal height was determined by GPS observations
HB1394.and is referenced to NAD 83.
HB1394
HB1394.The geoid height was determined by GEOID09.
HB1394
HB1394.The dynamic height is computed by dividing the NAVD 88
HB1394.geopotential number by the normal gravity value computed on the
HB1394.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
HB1394.degrees latitude (g = 980.6199 gals.).
HB1394
HB1394.The modeled gravity was interpolated from observed gravity values.
HB1394
HB1394;                   North        East     Units Scale Factor Converg.
HB1394;SPC MO E    -   228,494.237  309,287.266   MT  0.99997661  +0 24 50.3
HB1394;UTM  16     - 4,197,432.411  251,495.217   MT  1.00036068  -1 44 11.3
```

```
HB1394
HB1394!            - Elev Factor  x  Scale Factor =   Combined Factor
HB1394!SPC MO E    - 0.99998598  x  0.99997661  =   0.99996259
HB1394!UTM  16     - 0.99998598  x  1.00036068  =   1.00034666
HB1394
HB1394                        SUPERSEDED SURVEY CONTROL
HB1394
HB1394  ELLIP H (02/11/04)    89.315   (m)                          GP(       ) 4 1
HB1394  NAD 83(1997)-  37 53 25.83260(N)     089 49 33.44129(W)  AD(      ) B
HB1394  ELLIP H (03/31/98)    89.326   (m)                          GP(       ) 3 1
HB1394  NAVD 88 (03/31/98)  118.81    (m)             389.8    (f) LEVELING   3
HB1394  NGVD 29 (??/??/??)  118.742   (m)             389.57   (f) ADJUSTED   1 2
HB1394
HB1394.Superseded values are not recommended for survey control.
HB1394.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
HB1394.See file dsdata.txt to determine how the superseded data were derived.
HB1394
HB1394_U.S. NATIONAL GRID SPATIAL ADDRESS: 16SBG5149597432(NAD 83)
HB1394_MARKER: I = METAL ROD
HB1394_SETTING: 49 = STAINLESS STEEL ROD W/O SLEEVE (10 FT.+)
HB1394_SP_SET: STAINLESS STEEL ROD
HB1394_STAMPING: R 323 1981
HB1394_MARK LOGO: NGS
HB1394_PROJECTION: FLUSH
HB1394_MAGNETIC: N = NO MAGNETIC MATERIAL
HB1394_STABILITY: B = PROBABLY HOLD POSITION/ELEVATION WELL
HB1394_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
HB1394+SATELLITE: SATELLITE OBSERVATIONS - October 05, 2009
HB1394_ROD/PIPE-DEPTH: 4.30 meters
HB1394
HB1394  HISTORY      - Date       Condition        Report By
HB1394  HISTORY      - 1981       MONUMENTED       NGS
HB1394  HISTORY      - 19970226 GOOD              NGS
HB1394  HISTORY      - 19970626 GOOD              NGS
HB1394  HISTORY      - 19990830 GOOD              NGS
HB1394  HISTORY      - 20030724 GOOD              MODNR
HB1394  HISTORY      - 20030804 GOOD              MODNR
HB1394  HISTORY      - 20050315 GOOD              MODNR
HB1394  HISTORY      - 20091005 GOOD              MODNR
HB1394
HB1394                        STATION DESCRIPTION
HB1394
HB1394'DESCRIBED BY NATIONAL GEODETIC SURVEY 1981
HB1394'2.95 KM (1.85 MI) SOUTH FROM CHESTER.
HB1394'1.35 KILOMETERS (0.85 MILE) SOUTHWEST ALONG ILLINOIS STATE HIGHWAY 150
HB1394'AND MISSOURI STATE HIGHWAY 51 FROM THE JUNCTION REILY ROAD AND THE
HB1394'TOLL BOOTH OF THE BRIDGE IN CHESTER, THENCE 1.6 KILOMETERS (1.0 MILE)
HB1394'SOUTHEAST ALONG THE TOP OF THE MAIN LEVEE TO THE MARK ON THE RIGHT,
HB1394'4.42 METERS (14.5 FEET) SOUTHWEST OF THE CENTER OF THE LEVEE ROAD AND
HB1394'0.46 METER (1.5 FEET) SOUTHEAST OF A METAL WITNESS POST.
HB1394'THE MARK IS 0.46 METERS  NW FROM A WITNESS POST.
HB1394'THE MARK IS ABOVE LEVEL WITH TOP OF LEVEE.
HB1394
HB1394                        STATION RECOVERY (1997)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1997 (CSM)
HB1394'THE STATION IS LOCATED ABOUT 24.1 KM (14.95 MI) NORTH-NORTHEAST OF
HB1394'PERRYVILLE, MO.  AND ABOUT 3.2 KM (2.00 MI) SOUTH OF AND ACROSS THE
HB1394'MISSISSIPPI RIVER FROM CHESTER, IL., ALONG THE SOUTHWEST SIDE OF THE
HB1394'RIVER, IN THE GRASS NEAR THE TOP SOUTHWEST EDGE OF THE LEVEE ROAD,
HB1394'NEAR THE NORTHWEST END OF A FIELD ACCESS TRACK ROAD ON THE RIGHT, AND
HB1394'ACROSS THE LEVEE ROAD FROM A RIVER ACCESS ROAD ON THE LEFT.
HB1394'OWNERSHIP--ARMY CORPS OF ENGINEERS.  TO REACH THE STATION FROM THE
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I-11

HB1394'JUNCTION OF STATE HIGHWAY 51 AND THE LEVEE ROAD, NEAR THE SOUTHWEST
HB1394'END OF THE BRIDGE OVER THE MISSISSIPPI RIVER, SOUTHWEST OF CHESTER,
HB1394'IL., GO SOUTHEASTERLY, 1.64 KM (1.00 MI) ALONG THE LEVEE ROAD TO THE
HB1394'STATION ON THE RIGHT EDGE OF THE ROAD, JUST NORTHWEST OF A WITNESS
HB1394'POST. THE STATION IS 12.4 M (40.7 FT) SOUTHEAST OF A CULVERT RUNNING
HB1394'UNDER THE LEVEE, WITH THE ENDS OPEN TO A DITCH ON THE SOUTHWEST SIDE
HB1394'OF THE LEVEE AND A VALVE ON THE NORTHEAST SIDE OF THE LEVEE, 3.4 M
HB1394'(11.2 FT) SOUTHWEST OF THE LEVEE CENTER, 0.4 M (1.3 FT) NORTHWEST OF A
HB1394'WITNESS POST, AND THE STATION IS ABOUT 0.3 M (1.0 FT) BELOW THE ROAD
HB1394'LEVEL AND FLUSH WITH THE GROUND SURFACE. BY R.G. HAYES. NOTE--THE
HB1394'DATUM POINT IS A PUNCH MARK ON THE TOP CENTER OF A STAINLESS STEEL
HB1394'DATUM POINT WHICH IS CRIMPED TO THE TOP OF A STAINLESS STEEL ROD,
HB1394'DRIVEN TO A DEPTH OF 4.3 M, (14.1 FT) ENCASED IN A 5-INCH PVC PIPE
HB1394'WITH NGS LOGO CAP, SURROUNDED BY CONCRETE. ACCESS TO THE DATUM POINT
HB1394'IS THROUGH THE 5-INCH LOGO CAP.
HB1394
HB1394   STATION RECOVERY (1997)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1997 (CSM)
HB1394'RECOVERED AS DESCRIBED.
HB1394
HB1394   STATION RECOVERY (1999)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1999 (RB)
HB1394'RECOVERED AS DESCRIBED
HB1394'
HB1394
HB1394   STATION RECOVERY (2003)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2003 (WW)
HB1394'RECOVERED AS DESCRIBED.
HB1394
HB1394   STATION RECOVERY (2003)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2003 (BDC)
HB1394'RECOVERED IN GOOD CONDITION.
HB1394
HB1394   STATION RECOVERY (2005)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2005 (MJC)
HB1394'RECOVERED AS DESCRIBED. DESCRIPTION AND TO REACH ARE ADEQUATE.
HB1394
HB1394   STATION RECOVERY (2009)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2009 (MJC)
HB1394'
HB1394'THE STATION IS LOCATED IN T37N R11E, IN USS 440.
HB1394'
HB1394'IT IS 11 FT. SW OF THE CENTER OF LEVEE ROAD, 92.2 FT. WNW OF THE NORTH
HB1394'I-BEAM GATE POST, 95.1 FT. NW OF THE SOUTH I-BEAM GATE POST AND 1.0
HB1394'FT. NW OF A CARSONITE WITNESS POST.
HB1394'

I-9. NGS Data Sheet for L 289 (PID HB1377).

```
HB1377 ***********************************************************************
HB1377  CBN         -  This is a Cooperative Base Network Control Station.
HB1377  DESIGNATION -  L 289
HB1377  PID         -  HB1377
HB1377  STATE/COUNTY-  IL/JACKSON
HB1377  USGS QUAD   -  ROCKWOOD (1994)
HB1377
HB1377                     *CURRENT SURVEY CONTROL
HB1377  _____
HB1377* NAD 83(2007)-  37 45 51.31029(N)     089 39 20.89642(W)     ADJUSTED
HB1377* NAVD 88     -        116.393  (meters)    381.87  (feet)   ADJUSTED
HB1377  _____
HB1377  EPOCH DATE  -        2002.00
HB1377  X           -      30,328.286 (meters)                     COMP
HB1377  Y           -  -5,048,474.489 (meters)                     COMP
HB1377  Z           -   3,884,844.775 (meters)                     COMP
HB1377  LAPLACE CORR-        -0.37  (seconds)                      USDV2009
HB1377  ELLIP HEIGHT-        87.252 (meters)         (02/10/07) ADJUSTED
HB1377  GEOID HEIGHT-       -29.16  (meters)                       GEOID09
HB1377  DYNAMIC HT  -       116.313 (meters)    381.60  (feet)  COMP
HB1377
HB1377  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
HB1377  Type    PID   Designation                    North   East  Ellip
HB1377  --------------------------------------------------------------------
HB1377  NETWORK HB1377 L 289                          1.20    0.73   1.98
HB1377  --------------------------------------------------------------------
HB1377  MODELED GRAV-    979,933.4   (mgal)                       NAVD 88
HB1377
HB1377  VERT ORDER  -  FIRST      CLASS II
HB1377
HB1377.The horizontal coordinates were established by GPS observations
HB1377.and adjusted by the National Geodetic Survey in February 2007.
HB1377
HB1377.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
HB1377.See National Readjustment for more information.
HB1377.The horizontal coordinates are valid at the epoch date displayed above.
HB1377.The epoch date for horizontal control is a decimal equivalence
HB1377.of Year/Month/Day.
HB1377
HB1377.The orthometric height was determined by differential leveling
HB1377.and adjusted in June 1991.
HB1377
HB1377.The X, Y, and Z were computed from the position and the ellipsoidal ht.
HB1377
HB1377.The Laplace correction was computed from USDV2009 derived deflections.
HB1377
HB1377.The ellipsoidal height was determined by GPS observations
HB1377.and is referenced to NAD 83.
HB1377
HB1377.The geoid height was determined by GEOID09.
HB1377
HB1377.The dynamic height is computed by dividing the NAVD 88
HB1377.geopotential number by the normal gravity value computed on the
HB1377.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
HB1377.degrees latitude (g = 980.6199 gals.).
HB1377
HB1377.The modeled gravity was interpolated from observed gravity values.
HB1377
HB1377;                    North        East    Units Scale Factor Converg.
HB1377;SPC IL W   -  121,927.701  745,011.038  MT  0.99996612  +0 18 46.3
```

```
HB1377;SPC IL W      -   400,024.47 2,444,257.05   sFT  0.99996612   +0 18 46.3
HB1377;UTM 16        - 4,182,980.653  266,061.861   MT  1.00027412   -1 37 37.9
HB1377
HB1377!               - Elev Factor  x  Scale Factor =  Combined Factor
HB1377!SPC IL W       -   0.99998631  x   0.99996612  =  0.99995243
HB1377!UTM 16         -   0.99998631  x   1.00027412  =  1.00026042
HB1377
HB1377                       SUPERSEDED SURVEY CONTROL
HB1377
HB1377  NAD 83(1997)- 37 45 51.31026(N)     089 39 20.89637(W) AD(      ) A
HB1377  ELLIP H (09/15/03)  87.253  (m)                        GP(      ) 4 1
HB1377  NGVD 29 (??/??/??)  116.316 (m)            381.61  (f) ADJUSTED  1 2
HB1377
HB1377.Superseded values are not recommended for survey control.
HB1377.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
HB1377.See file dsdata.txt to determine how the superseded data were derived.
HB1377
HB1377_U.S. NATIONAL GRID SPATIAL ADDRESS: 16SBG6606182980(NAD 83)
HB1377_MARKER: I = METAL ROD
HB1377_SETTING: 59 = STAINLESS STEEL ROD IN SLEEVE (10 FT.+)
HB1377_SP_SET: STAINLESS STEEL ROD IN SLEEVE
HB1377_STAMPING: L 289 1981
HB1377_MARK LOGO: NGS
HB1377_PROJECTION: FLUSH
HB1377_MAGNETIC: N = NO MAGNETIC MATERIAL
HB1377_STABILITY: A = MOST RELIABLE AND EXPECTED TO HOLD
HB1377+STABILITY: POSITION/ELEVATION WELL
HB1377_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
HB1377+SATELLITE: SATELLITE OBSERVATIONS - March 08, 2006
HB1377_ROD/PIPE-DEPTH: 21.9 meters
HB1377_SLEEVE-DEPTH : 6.10 meters
HB1377
HB1377  HISTORY    - Date      Condition        Report By
HB1377  HISTORY    - 1981      MONUMENTED       NGS
HB1377  HISTORY    - 20020805 GOOD             NGS
HB1377  HISTORY    - 20060308 GOOD             ILDT
HB1377
HB1377                       STATION DESCRIPTION
HB1377
HB1377'DESCRIBED BY NATIONAL GEODETIC SURVEY 1981
HB1377'8.1 KM (5.05 MI) SOUTH FROM CORA.
HB1377'8.1 KILOMETERS (5.05 MILES) SOUTH ALONG THE TOP OF THE MAIN LEVEE FROM
HB1377'THE INTERSECTION OF THE MISSOURI PACIFIC RAILROAD IN CORA TO A BEND IN
HB1377'THE LEVEE AND THE MARK ON THE LEFT, AT THE JUNCTION OF A SPUR LEVEE
HB1377'LEADING SOUTHEAST, IN LINE WITH THE CENTER OF THE SPUR LEVEE,
HB1377'4.57 METERS (15.0 FEET) NORTH OF THE CENTER OF THE LEVEE ROAD AND
HB1377'0.46 METERS (1.5 FEET) EAST OF A METAL WITNESS POST.
HB1377'THE MARK IS 0.46 METERS  W FROM A WITNESS POST.
HB1377'THE MARK IS 0.15 M BELOW TOP OF LEVEE.
HB1377
HB1377                       STATION RECOVERY (2002)
HB1377
HB1377'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 2002 (BE)
HB1377'RECOVERED AS DESCRIBED
HB1377'
HB1377'
HB1377
HB1377                       STATION RECOVERY (2006)
HB1377
HB1377'RECOVERY NOTE BY ILLINOIS DE
```

APPENDIX J

Establishing NSRS Elevations on 15 Dam and Reservoir Projects in Pittsburgh District

J-1. General. The following example outlines the survey actions performed by Pittsburgh District to establish NAVD88 elevations on 15 dam and reservoir projects throughout the District. The procedures used in this example project are also applicable to levee projects extending over large geographical areas. This work outlined below was performed by Terrasurv, Inc. under a contract to the Pittsburgh District. Adjusted GPS observations, from which NAVD 88 elevations were derived, were submitted to NGS for inclusion in the in the NSRS. NSRS referenced NAVD88 elevations were established at Primary Project Control Points (PPCPs) at each dam and reservoir site.

J-2. Background. As part of a Corps-wide review of project datums initiated in 2007 (i.e., the Comprehensive Evaluation of Project Datums or CEPD), the Pittsburgh District undertook an assessment of all their projects. Part of this effort was to evaluate the origin and accuracy of the vertical datums in use at each of the sixteen dam and reservoir projects within the District's civil works area of responsibility. This CEPD review determined that none of these projects had a documented connection to the NSRS. While all of the projects nominally had NGVD29 elevations, the source and accuracy of many was in question. In addition, the data was not in a format conducive to updating to NAVD88.

    a. East Branch Project. The Clarion River East Branch Reservoir was used as a pilot project. GPS survey methods were used on the East Branch project to provide ties to NSRS bench marks. The data was then formatted and submitted to the NGS for inclusion in the NSRS. Additional details on this specific project are outlined in Appendix F.

    b. Remaining reservoir projects. After completing the East Branch Project, the District decided to implement a similar process to establish an NSRS referenced PPCP at each of the remaining 15 reservoir projects. These projects encompassed an area of approximately 26,000 square miles in western Pennsylvania, a small portion of western New York, eastern Ohio, and northern West Virginia.

J-3. Project Description. Figure J-1 depicts the 16 dam and reservoir projects contained in Pittsburg District's civil works boundary. The geodetic survey scheme shown on the right side of the figure was developed to establish vertical (and horizontal) control on an existing primary monitoring bench mark at each of the remaining 15 project sites. These geodetic control surveys were performed during annual O&M programmed deformation monitoring surveys at each dam—significantly reducing costs in updating datum references. (Pittsburgh District estimates the cost to perform the NSRS connection at each site was approximately $8,000). The observed GPS baseline data was then processed, adjusted, and submitted to NGS for inclusion in the NSRS. The District's 23 lock and dam projects shown on Figure J-1 were not included in this overall project. These projects could be linked to this reservoir control network at some future date.
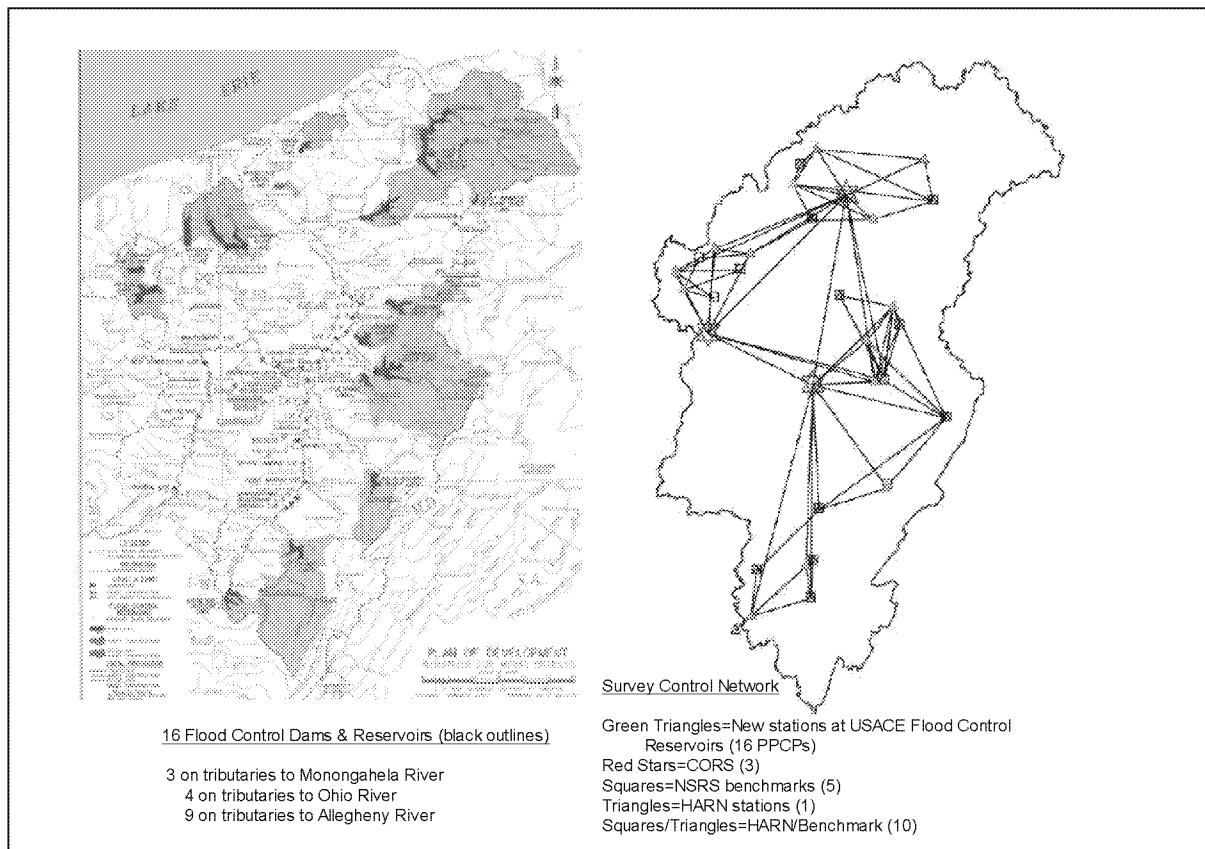
16 Flood Control Dams & Reservoirs (black outlines)

3 on tributaries to Monongahela River
4 on tributaries to Ohio River
9 on tributaries to Allegheny River

Survey Control Network

Green Triangles=New stations at USACE Flood Control
　　　Reservoirs (16 PPCPs)
Red Stars=CORS (3)
Squares=NSRS benchmarks (5)
Triangles=HARN stations (1)
Squares/Triangles=HARN/Benchmark (10)

Figure J-1. GPS data collection network used to establish primary control at 16 dam and reservoir projects in Pittsburg District.

J-4. <u>GPS Network Decision</u>. The A-E contractor (Terrasurv, Inc.) had been performing annual deformation surveys at the reservoirs since 2005. Many of these prior surveys utilized GPS to provide external control for the deformation analysis. Although CORS/OPUS observations at each site would have provided adequate NAVD88 elevations (accurate to $< \pm 0.25$ ft), it was decided that previously observed GPS baselines could be readily incorporated into a higher-accuracy geodetic network encompassing all projects. This network would effectively link all the District's projects to the NSRS and to one another. Accordingly, GPS observations from 2005, 2006, and 2007 were examined to extract observations that would be useful in the current network survey. A "Primary Project Control Point" (PPCP) was selected at each reservoir—typically one of the structural deformation monitoring points (pedestals). Some reservoirs had a second point included in the network due to multiple GPS observation in previous years.

J-5. <u>Existing NSRS Control</u>. Because the primary purpose of the project was to establish NAVD88 elevations at each reservoir, emphasis was placed on using NSRS marks that were stable bench marks. Because there were numerous ties to CORS, horizontal accuracy of the existing NSRS stations was secondary. Most of the existing NSRS control stations used in Pennsylvania and West Virginia were both horizontal and vertical control. In Ohio, it was not possible to find any such stations in the vicinity of the reservoirs, therefore three vertical-only

bench marks were used there, along with a local CORS. Table J-1 lists the existing NSRS control used on this project.

Table J-1. Existing NSRS Control.

Station Name	PID	Horizontal Ellipsoidal Accuracy	Ortho Order	State	Comments	Stability	Nearest Reservoir
R HAYES	AA9347	0.65/1.04	GPS	WV	HARN	D	Stonewall
C 317	JW1099	1.22/2.12	I-2	WV	HARN/BM	A	Tygart
P 322	JW1091	1.04/1.67	I-2	WV	HARN/BM	A	Tygart/Stonewall
W 319	JX1767	1.18/1.78	I-2	WV	HARN/BM	B	Stonewall
A 121	JW0568	----	I-2	PA	V only	D	Youghiogheny
T 404	KX1902	1.30/2.27	I-2	PA	HARN/BM	B	Mahoning
E 402	KX1814	0.77/1.33	I-2	PA	HARN/BM	B	Mahoning/Crooked Creek
G 316	KX0579	0.71/1.35	II-0	PA	HARN/BM	D	Youghiogheny
E 313	JW1043	0.61/1.16	I-2	WV	HARN/BM	B	Youghiogheny/Tygart
TTS 64 K	MA0735	0.31/0.51	II-0	PA	HARN/BM	A	East Branch/Kinzua
D 406	MB1777	0.86/1.53	I-2	PA	HARN/BM	B	Woodcock/Union Cty
E 408	MA1819	0.77/1.39	I-2	PA	HARN/BM	B	Tionesta
D 156	MB0852	----	2-0	OH	V only	B	Mosquito/Kirwan
R 147	KY1127	----	2-0	OH	V only	B	Berlin
AAA	MB0984	----	2-0	OH	V only	B	Shenango
PAPT	DF7986	CORS	CORS	PA	CORS		
UPTC	AI8355	CORS	CORS	PA	CORS		
LSBN	DF4054	CORS	CORS	OH	CORS		

J-6. <u>Conemaugh Dam Bench Marks</u>. In addition to the existing NSRS marks described above, two stable NSRS bench marks (PID KX1047 and PID KX1045) were recovered near the Conemaugh Dam, and a short level line was run between them to establish a new mark which was capable of being occupied by GPS receivers. This level line was run twice with invar bar code rods, once in 2005 and again in 2008. The misclosure between these marks relative to the published elevations was 0.0065 m. PBM 52A appeared to have been disturbed.

J-7. <u>Datums</u>. The horizontal datum used for this network project is the NAD83 (NSRS 2007). The vertical datum is NAVD88. The geoid model used was GEOID 2003.

J-8. <u>GPS Observations</u>. Observations were made over a time span from May of 2005 through January of 2009. All observations were made using Trimble dual frequency receivers.

    a. The receivers used included the following:

    (1) two R8 GNSS models, with integral antennas.

(2) one R6 model, with integral antenna.

(3) two 5700 models, one with a Zephyr Geodetic antenna and the other with a Zephyr antenna.

(4) one 4800 model, with integral antenna.

(5) one 4700 model, with a microcenter L1/L2 antenna without ground plane.

(6) one 4400 model with a compact L1/L2 antenna, without ground plane.

b. Data was obtained from the CORS sessions with a sufficient amount of data to ensure an accurate solution. The survey pedestals were occupied by placing a standard survey tribrach onto the stainless steel plate. The 5/8" rod typically protrudes about 12 mm above the plate. The height of instrument measurement is with respect to the top of this rod. Most of the ground points (disks) were occupied using fixed height tripods as shown in Figure J-2. A few were occupied with standard survey tripods. All height of instrument measurements were reduced to vertical values from the mark to the Antenna Reference Point (ARP), which is the bottom of the antenna housing.
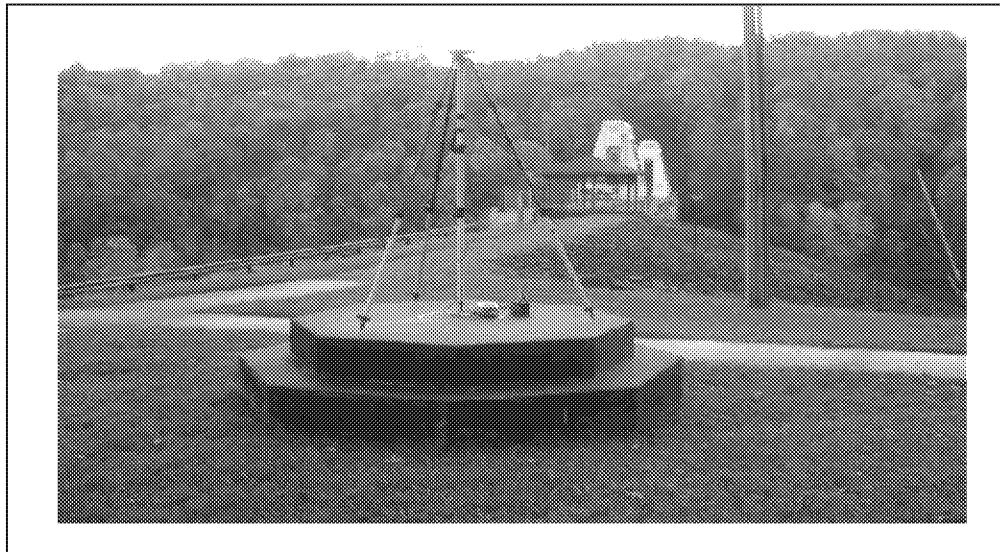

Figure J-2. GPS data collection at a primary structural monitoring point (PPCP) near the dam.

J-9. GPS Data Processing. The GPS observables were downloaded from the receivers and processed using the Weighted Ambiguity Vector Estimator (WAVE) processor in Trimble Geomatics Office, version 1.63. The precise ephemeris (IGS Rapid) was used for processing baselines. Not all baselines were processed—several of the lines had higher than normal statistics, indicating the presence of noise in the data. Many of the baselines were measured

twice, and therefore had an independent verification of correct integer ambiguity resolution. Seven processed baselines were disabled in the data analysis phase. Many of the baselines were measured in more than one session. The independently determined baseline components were transformed from an Earth Centered Earth Fixed (ECEF) system to a local horizon system (N-E-U). All of the new reservoir project stations were occupied at least once. Two existing NSRS control stations, "R HAYES" and "D 406," were occupied a single time.

J-10. Least Squares Adjustments. The data was adjusted using ADJUST, a least squares adjustment program from the NGS. The occupation information was processed to form an ADJUST "B-file." The processed baselines were parsed to form an input file in the ADJUST "G-file" format.

a. Minimally constrained NSRS adjustment. The first adjustment constrained the CORS "PAPT ARP" station to the published NAD83 (epoch 2002.0) position (latitude, longitude, and ellipsoidal height). The resultant standard deviation of unit weight was 6.75. The misclosures at the other NSRS stations and the other two CORS used are shown in Table J-2.

Table J-2. Minimally Constrained Adjustment of NSRS Control Holding "PAPT ARP" Fixed.

Station	Azimuth	Distance (m)	Δ Ellipsoidal Height (m)
R HAYES	196	0.007	-0.010
C 317	315	0.003	-0.012
P 322	188	0.008	-0.031
W 319	214	0.004	0.009
T 404	116	0.013	0.018
E 402	242	0.008	-0.010
G 316	200	0.007	-0.011
E 313	31	0.002	0.028
TTS 64 K	255	0.016	-0.002
D 406	238	0.009	0.005
E 408	251	0.007	0.012
UPTC ARP	129	0.005	-0.007
LSBN ARP	272	0.013	0.002

The misclosures on the above NSRS control stations were considered excellent, especially given the network spans an area of about 65,000 square kilometers (24,000 square miles).

b. Minimally Constrained Orthometric Height Adjustment. Next, a minimally constrained adjustment was done holding the published NAVD88 orthometric height of bench mark "T 404" (PID KX1902), and "PAPT ARP" fixed horizontally. The misclosures at the other NSRS marks with published orthometric heights are shown in Table J-3 and graphically on Figure J-3.

Table J-3. Minimally Constrained Adjustment Holding "T 404" (V) and "PAPT ARP" (3D).

Station	Δ Ortho Height
R HAYES (GPS Derived Ortho Height)	0.010 m
CONEMAUGH BRIDGE	
(leveling by Terrasurv)	-0.022 m
C 317	-0.007 m
P 322	0.000 m
W 319	0.030 m
A 121 (checked by levels to adjacent BM)	0.066 m
E 402	-0.008 m
G 316	-0.030 m
E 313	0.038 m
TTS 64 K	-0.006 m
D 406	-0.001 m
E 408	0.015 m
D 156	-0.015 m
R 147	0.037 m
AAA	0.027 m

c. The bench mark with the highest misclosure, "A 121" (0.066 m), was checked against an adjacent NSRS mark using precise leveling techniques and equipment (Second Order procedures). This mark had four repeat baselines, two in 2005 and two in 2008. The vertical component of the baseline from YOUGHIOGHENY M1 to A 121 was - 49.818 m, - 49.806 m, - 49.815 m, and - 49.800 m, with a range of ± 0.010 m about the mean value of - 49.810 m. Therefore, this mark is believed to be stable and the lines going to it are of high accuracy. This mark is located on the eastern side of two mountain ridges (Chestnut and Laurel Ridges), whereas the rest of the project is located on the western side of the ridges. The geoid model used (2003) may be affected by these ridges. In order to ensure an accurate GPS derived orthometric height at the nearby Youghiogheny Reservoir, this mark was included as a constraint in the final constrained orthometric height adjustment. All of the other existing marks were within ± 0.037 m in the vertical (orthometric height) component.
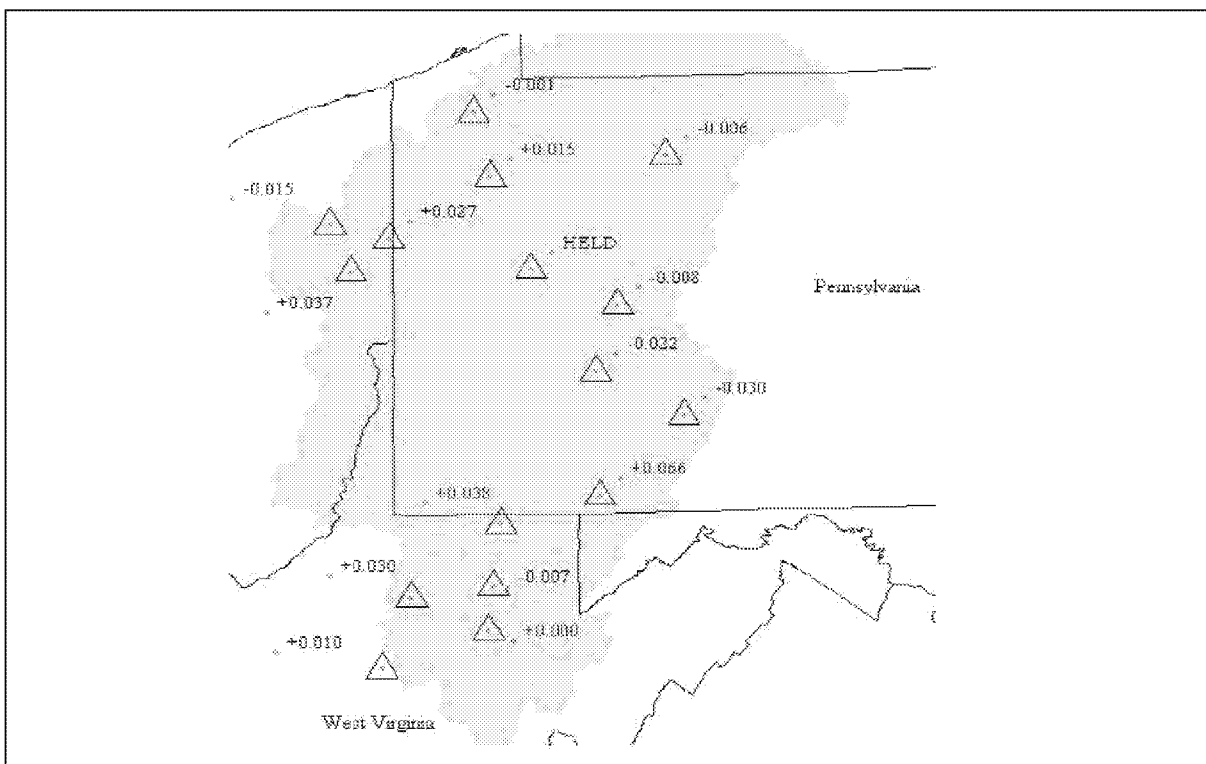
Figure J-3. Misclosures resulting from a minimally constrained vertical adjustment constraining height on a HARN bench mark near the center of the project. (Misclosures shown in meters)

d. Constrained adjustment (ellipsoid heights). The next adjustment was a fully constrained adjustment that held the three CORS and the eleven existing NSRS stations with published NAD1983 (NSRS2007) latitudes, longitude, and ellipsoidal heights. This adjustment had a standard deviation of unit weight of 8.165. This adjustment provided the adjusted latitudes, longitudes, and ellipsoidal heights for the network.

e. Constrained adjustment (orthometric heights). The final constrained adjustment held CORS "PAPT ARP" fixed horizontally, and fifteen stations with leveled NAVD88 orthometric heights were constrained vertically in the NGS ADJUST program. The standard deviation of unit weight was 7.20 and the residual RMS was ± 0.021 m. This adjustment provided the NAVD88 GPS derived orthometric heights for the network. The final adjustment results are listed in Table J-4.

Table J-4. Adjusted Coordinates – NAD83 NSRS 2007/NAVD88.

Station Name	Latitude	Longitude	Ellipsoid Height (m)	NAVD88 Height (m)
TYGART ASE2	39°18'47.16840" N	80°01'45.99898" W	345.026	377.139
CROOKED CK M2	40°42'53.07204" N	79°30'49.51338" W	257.176	290.573
MOSQUITO M4	41°18'06.84178" N	80°45'09.48816" W	246.597	280.407
STONEWALL	39°00'15.54934" N	80°28'24.35241" W	283.918	316.238
R HAYES	38°54'47.26507" N	80°35'44.73360" W	336.208	368.608
YOUGHIOGHENY M1	39°47'55.96185" N	79°22'02.18435" W	424.933	456.671
CONEMAUGH BR	40°27'42.15164" N	79°22'02.71448" W	245.331	278.377
CONEMAUGH M3	40°28'05.77660" N	79°22'29.30277" W	286.840	319.890
LOYALHANNA	40°27'29.96032" N	79°27'09.34669" W	269.118	302.254
TIONESTA	41°28'24.00763" N	79°26'35.72847" W	322.841	355.812
UNION CITY M1	41°55'14.71075" N	79°54'11.33916" W	362.504	396.550
UNION CITY M5	41°55'14.56028" N	79°54'16.09282" W	371.722	405.770
KIRWAN M1	41°08'47.32372" N	81°04'22.71305" W	275.309	308.988
C 317	39°20'38.54354" N	79°58'03.73837" W	325.837	357.872
BERLIN M5	41°02'35.39945" N	81°00'32.70465" W	285.874	319.559
P 322	39°06'38.37991" N	79°59'42.18154" W	525.250	556.874
W 319	39°17'11.27553" N	80°25'31.19084" W	332.512	365.178
A 121	39°48'45.98850" N	79°21'33.77400" W	375.121	406.837
MAHONING	40°55'28.22823" N	79°16'58.28579" W	356.660	389.675
KINZUA M1	41°50'31.50495" N	79°00'02.33348" W	387.903	420.339
WOODCOCK	41°42'05.73148" N	80°06'07.07176" W	339.158	372.986
SHENANGO	41°15'54.80683" N	80°27'45.43598" W	250.654	284.444
KIRWAN	41°09'26.19271" N	81°04'41.85584" W	253.577	287.257
MOSQUITO	41°17'59.95573" N	80°45'30.50304" W	245.336	279.148
T 404	40°59'41.81625" N	79°43'32.90556" W	336.103	369.602
E 402	40°48'07.37960" N	79°14'00.67131" W	338.584	371.572
G 316	40°13'30.90049" N	78°52'17.31696" W	554.614	586.700
E 313	39°40'10.80731" N	79°55'09.32013" W	345.395	377.956
TTS 64 K	41°34'50.57939" N	78°56'06.94800" W	541.539	573.625
D 406	41°49'17.00008" N	80°02'40.75526" W	314.420	348.439
E 408	41°28'42.27988" N	79°57'06.99473" W	356.203	389.618
D 156	41°14'17.11949" N	80°52'54.65838" W	239.435	273.263
R 147	40°59'17.67806" N	80°45'42.03236" W	310.755	344.636
AAA	41°09'49.81358" N	80°32'32.18072" W	247.341	281.222
PAPT ARP	40°26'40.25568" N	79°57'32.12989" W	313.698	347.441
UPTC ARP	41°37'43.70199" N	79°39'50.62173" W	343.186	376.455
LSBN ARP	40°46'08.93523" N	80°48'37.47136" W	314.449	348.434

J-11. Supplemental Ties to LPCPs. At each reservoir project, supplemental ties were made to local project control points around the project. In addition, ties were made to any water level or pool gages at the sites. Differential levels were run from primary PPCP mark to secondary marks using a DiNi 12 digital level. Levels runs to outflow gage reference PBMs were run if feasible, otherwise GPS was used to transfer elevations from the primary PPCP station to the outflow bench mark. Figures J-4 illustrates connections from the primary PPCP to supplemental bench marks at a dam and the down stream outflow PBM reference point. Figure J-5 shows leveling connections to a typical reservoir pool gage.

J-12. Summary. The geodetic control network met NGS height modernization standards and established consistent NAVD 1988 vertical control for the reservoirs in the Pittsburgh District. An overall accuracy of ±0.03 m was achieved in all three dimensions in a network covering the entire 26,000 sq mile district.
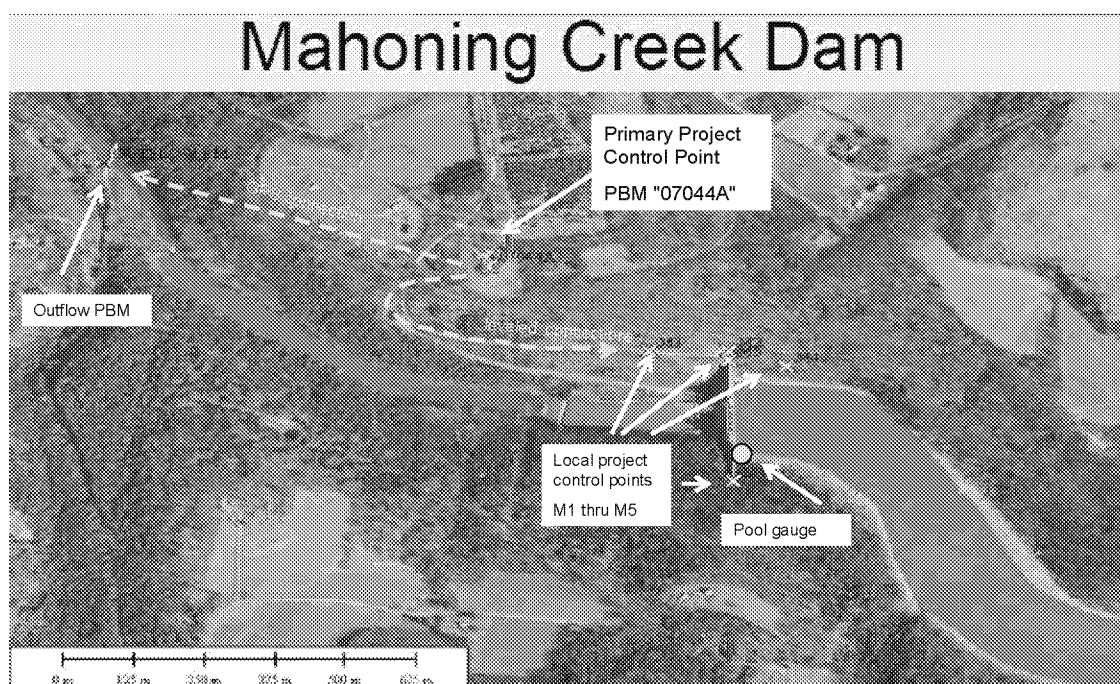
Figure J-4. Differential level connections from the PPCP to supplemental PBMs around Mahoning Creek Dam. The outfall PBM was connected by static GPS from the PPCP.
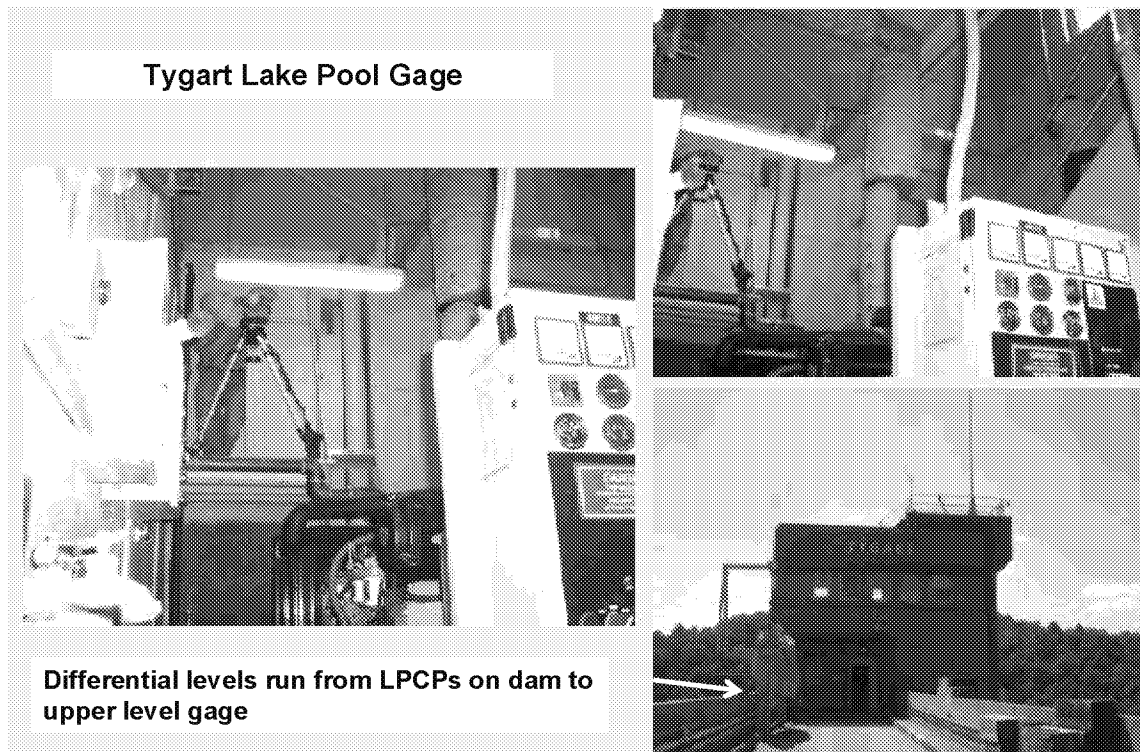
Figure J-5. Differential leveling connections to the pool gage at
Tygart Lake Dam and Reservoir.

APPENDIX K

Lake Superior Navigation Project Referenced to IGLD85—Ontonagon Harbor, Michigan
(Detroit District)

K-1. Purpose. This appendix contains an example of a Detroit District harbor project in Lake Superior (Figure K-1) that has been adequately referenced to the current NSRS orthometric datum and to the local reference water level datum in Lake Superior (IGLD85). The project consists of two breakwaters that have established reference baselines with local control on an IGLD85 vertical datum that is tied to a "Dynamic Height" reference plane used in the Great Lakes region—see Figure K-2. NOAA water level gages and bench marks in the Great Lakes are referenced to both orthometric heights (NAVD88) and dynamic heights (IGLD85). An overview on the establishment of IGLD85 and a list of IGLD reference datums throughout the Great Lakes and Connecting Waterways is at the end of this appendix.
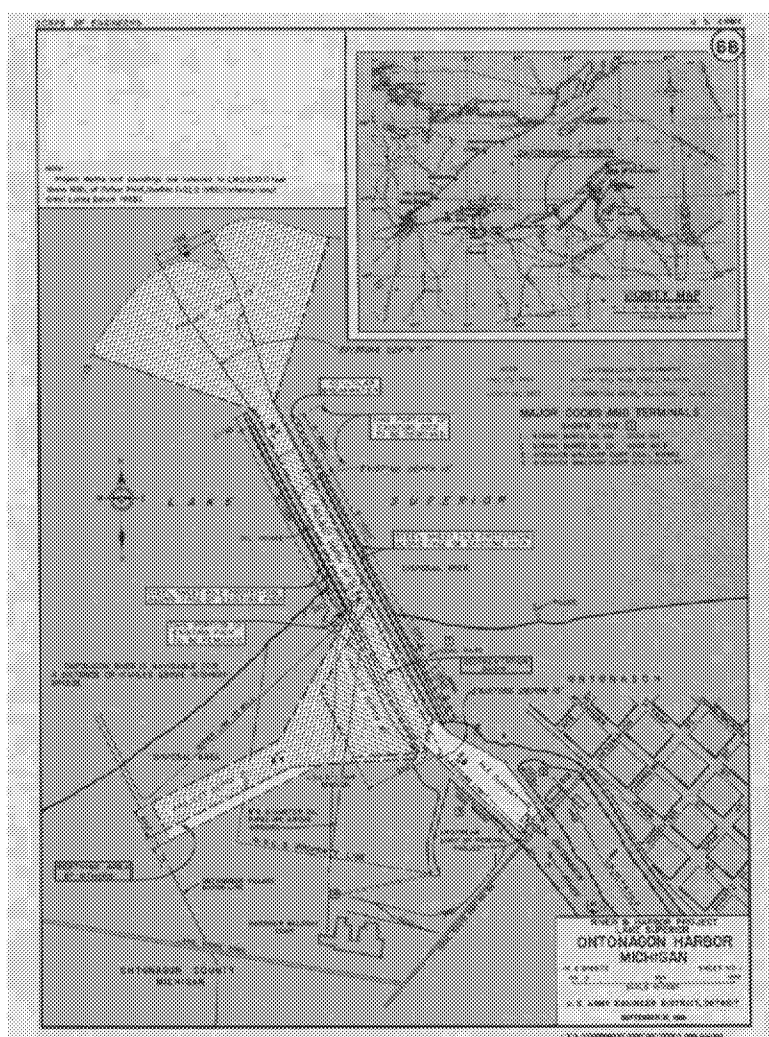
Figure K-1. Ontonagon Harbor, Michigan – Project Map.

K-1

K-2. Project Description.

ONTONAGON HARBOR, MICHIGAN

CONDITION OF IMPROVEMENT 30 SEPTEMBER 1986

EXISTING PROJECT: Authorized by the R&H Acts of 2 March 1867, 23 June 1874, 13 June 1902, 2 March 1907, 3 March 1909, 26 August 1937 and Act of 1962. Earlier authorizations (1910 and 1937) provide for a flared lake approach channel about 850 feet long to deep water and 16 feet deep with depths narrowing from 400 feet at the outer end to 150 feet opposite the outer end of the west pier; a channel between the piers 150 feet wide, 17 feet deep in the outer 250 feet, and 15 feet deep in the inner 2,200 feet; an inner basin 12 feet deep and 900 feet long, extending between lines 50 feet from the existing wharves on each side of the river, the maximum width being 200 feet; and the maintenance of this channel, the basin, and the east and west entrance piers which are 2,315 feet and 2,563 feet long, respectively.

Figure K-2. Ontonagon Harbor Michigan – Structure and federal navigation channel.

K-3. <u>Connections to NSRS and Tidal Datum References</u>. The Tidal BM established for the project is BM "D 135" as shown on Figure K-3. This point is published in the NSRS and has observed NAD83/GRS80 ellipsoid height observations, as excerpted from the NGS datasheet in Figure K-3.

```
RL0728  CBN          -   This is a Cooperative Base Network Control Station.
RL0728  DESIGNATION -   D 135
RL0728  PID          -   RL0728
RL0728  STATE/COUNTY-   MI/ONTONAGON
RL0728  USGS QUAD    -
RL0728
RL0728                        *CURRENT SURVEY CONTROL
RL0728
RL0728* NAD 83(2007)-  46 52 20.23258(N)      089 19 19.44724(W)      ADJUSTED
RL0728* NAVD 88      -       186.877  (meters)     613.11   (feet)   ADJUSTED
RL0728
RL0728  EPOCH DATE   -        2002.00
RL0728  X            -       51,683.527 (meters)                      COMP
RL0728  Y            -   -4,367,861.008 (meters)                      COMP
RL0728  Z            -    4,632,183.750 (meters)                      COMP
RL0728  LAPLACE CORR-          -4.82  (seconds)                     USDV2009
RL0728  ELLIP HEIGHT-         155.401 (meters)          (02/10/07) ADJUSTED
RL0728  GEOID HEIGHT-         -31.44  (meters)                      GEOID09
RL0728  DYNAMIC HT   -        186.907 (meters)     613.21  (feet)   COMP
RL0728
RL0728  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
RL0728  Type   PID   Designation                     North  East  Ellip
RL0728  -----------------------------------------------------------------
RL0728  NETWORK RL0728 D 135                           1.55   1.12   5.21
RL0728  -----------------------------------------------------------------
RL0728  MODELED GRAV-    980,770.6   (mgal)                        NAVD 88
RL0728
RL0728  VERT ORDER  -  FIRST     CLASS II
```

Figure K-3. Ontonagon Harbor Michigan – Tidal NGS BM D 135 Datasheet.

The 613.11 ft NAVD88 elevation of D135 is based on adjusted leveling observations. The estimated 95% confidence of the ellipsoid height is about 5 cm. Thus, this is an excellent point for use as an NOS gaging station from which all supplemental surveys can be referenced.

    a. Local PBM and TBM control. Figure K-4 lists the local reference PBMs and TBMs that are used to control structure cross-section monitoring. All USACE monuments are run in level differential levels loops to the two known tidal bench marks. All elevation measurements were relative to BM"D 135" on the NAVD88 reference datum. All reported elevations on design surveys and channel depth surveys are referenced to IGLD85 datum.

    b. IGLD85. IGLD85 is expressed as a dynamic height. Informally, this could also be considered as a height equivalent above mean sea level, based on work required to raise a unit mass. IGLD85 is also based on an adopted elevation at Point Rimouski/Father's Point. And, IGLD85 is realized as mean water levels at a set of master water level stations on the Great

Lakes. Due to various observational, dynamical, and satiric effects, there will be slight departures between a dynamic height and an IGLD85 height. These departures are known as hydraulic correctors, and are part of the NAVD88/IGLD85 datum transformation.

c. Conversion from NAVD88 Dynamic Height to IGLD85. The survey specifications required that structure profile data and all topographic be referenced on IGLD85, reported as true elevations. However soundings are reported as negative numbers in relationship to the datum of IGLD85 601.1 ft for Lake Superior. Thus, a (-) 26.5 ft depth reported on a condition survey is equal to an IGLD85 elevation of (601.1 - 26.5) = 574.6 ft.

(1) The water surfaces of all connecting channels and other rivers on the Great Lakes are considered to be sloping surfaces. Therefore, their Hydraulic Corrector is zero.

(2) The "Hydraulic Corrector" at each gage site on the lake has been incorporated into the data retrieval and storage process. As such, water level information is stored mechanically or electronically, at the NOAA CO-OPS or the Canadian Department of Fisheries and Oceans (DFO). Water elevations are referenced to IGLD85 and do not require any further adjustment.

(3) Hydraulic Correctors for several harbors in Lake Superior are listed below in Table K-1. IGLD85 vertical datum is based on calculated and interpolated corrections to be applied to Dynamic Heights for the Great Lakes region.

Table K-1. Lake Superior Hydraulic Correctors for IGLD85 (in meters).

PROJECT	LOCATION	HC	IHC
Duluth-Superior MN-WI	Lake Superior	0.3	--
Ontonagon Harbor MI		0.2	--
Grand Travis Bay MI		--	0.1
Saxon Harbor WI		--	0.2
IHC = Interpolated from established Hydraulic Correctors HC = Hydraulic Correctors from "Establishment of International Great Lakes Datum" December 1995			

INTERNATIONAL GREAT LAKES DATUM (1985)
Tabulation of Primary Bench Mark,
ONTONAGON

Primary PBM NO 2		Hydraulic Corrector	0.049 m
IGLD85 elev 185.443 m		Diff IGLD85-IGLD55	0.323 m

d. NOAA gage Ontonagon. The NOAA CO-OPS station data for the Ontonagon gage is shown in Figures K-4 and K-5.
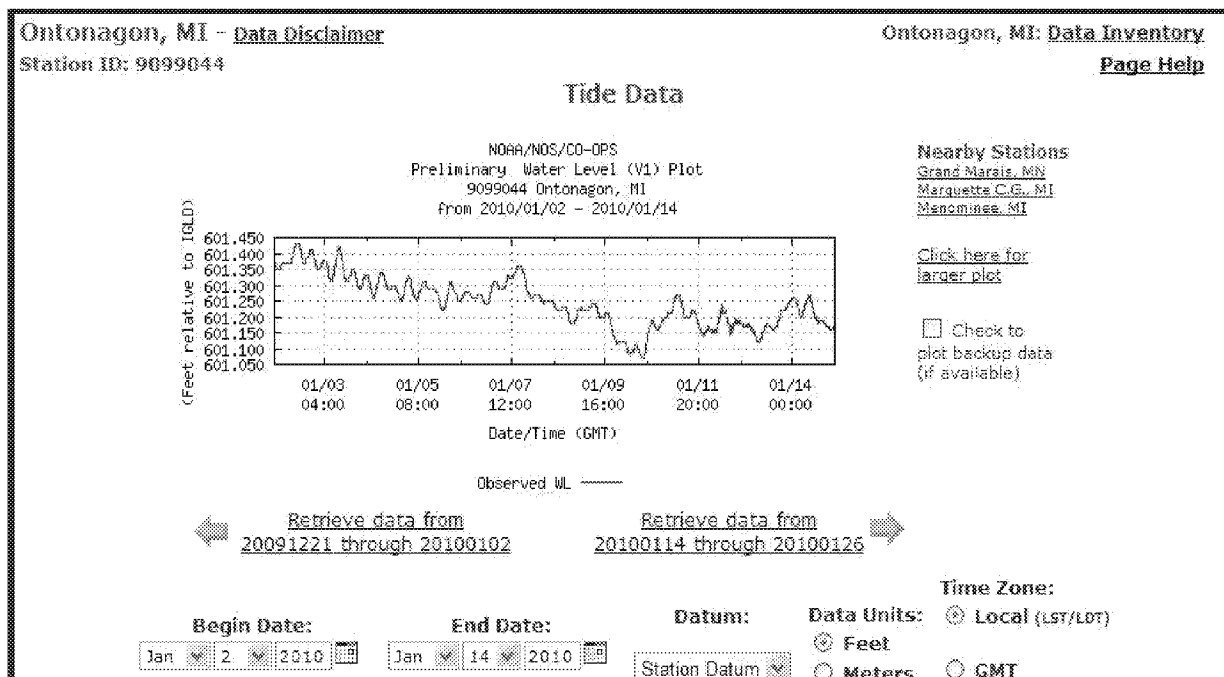
Figure K-4. Published NOAA/CO-OPS gage data in IGLD85 per CO-OPS web site.

Figure K-5. Published NOAA/CO-OPS gage data in IGLD85 per WEB site in IGLD85 in Local Standard Time (LST) in six-minute intervals.

e. Sounding corrections. Based on the gage data in Figure K-5 for the date shown (2 Jan 10), the water level at Ontonagon is 0.27 ft above IGLD85 (601.1 ft). To correct all sounding data observed on 2 Jan 10 to the IGLD85 depth, one would subtract 0.27 ft from all observed soundings. The difference between NAVD88 and IGLD85 is assumed the same throughout this small project site. Figure K-6 illustrates survey depths referenced to IGLD85 on a Project Condition Survey of Ontonagon Harbor. Figures K-7a and K-7b are examples of design placement grades on the IGLD85 reference datum.

Figure K-6. Hydrographic condition survey of federal navigation channel with soundings referenced to IGLD85 (601.1 ft) Datum.
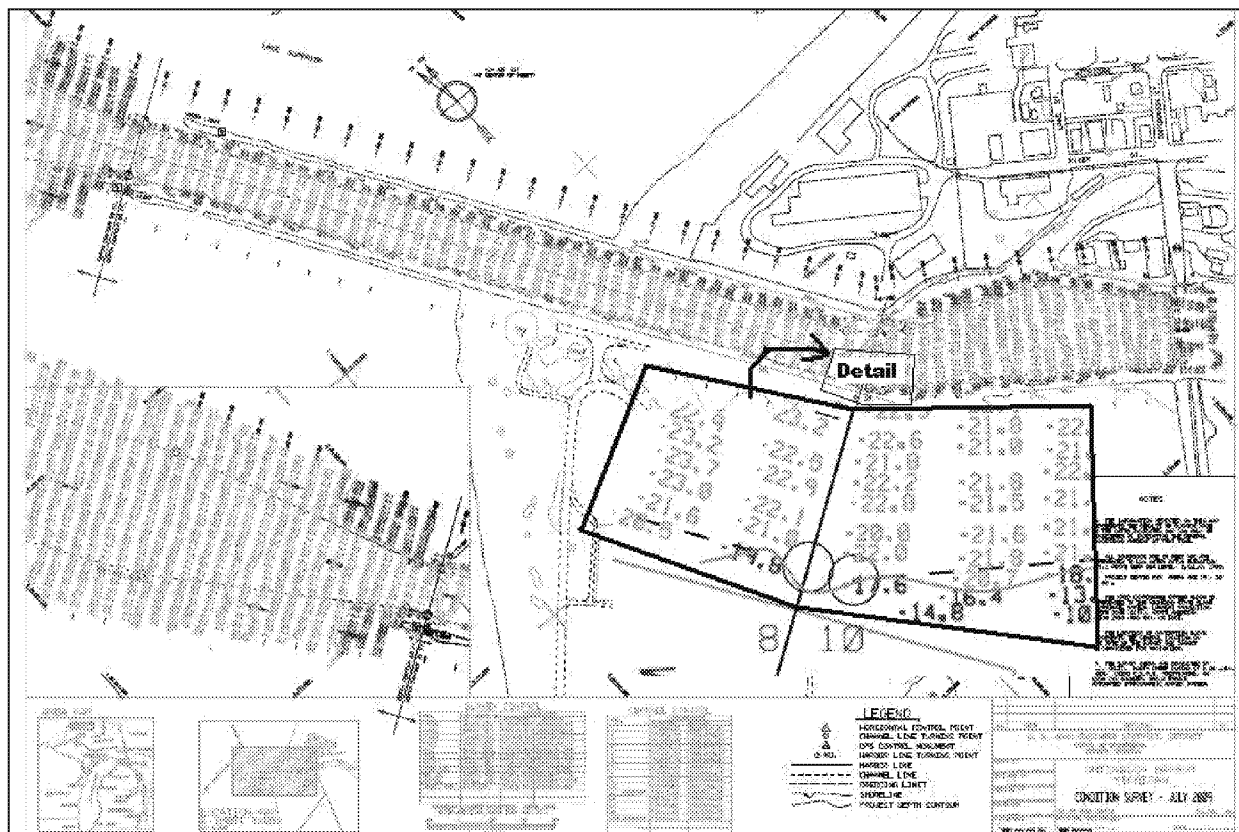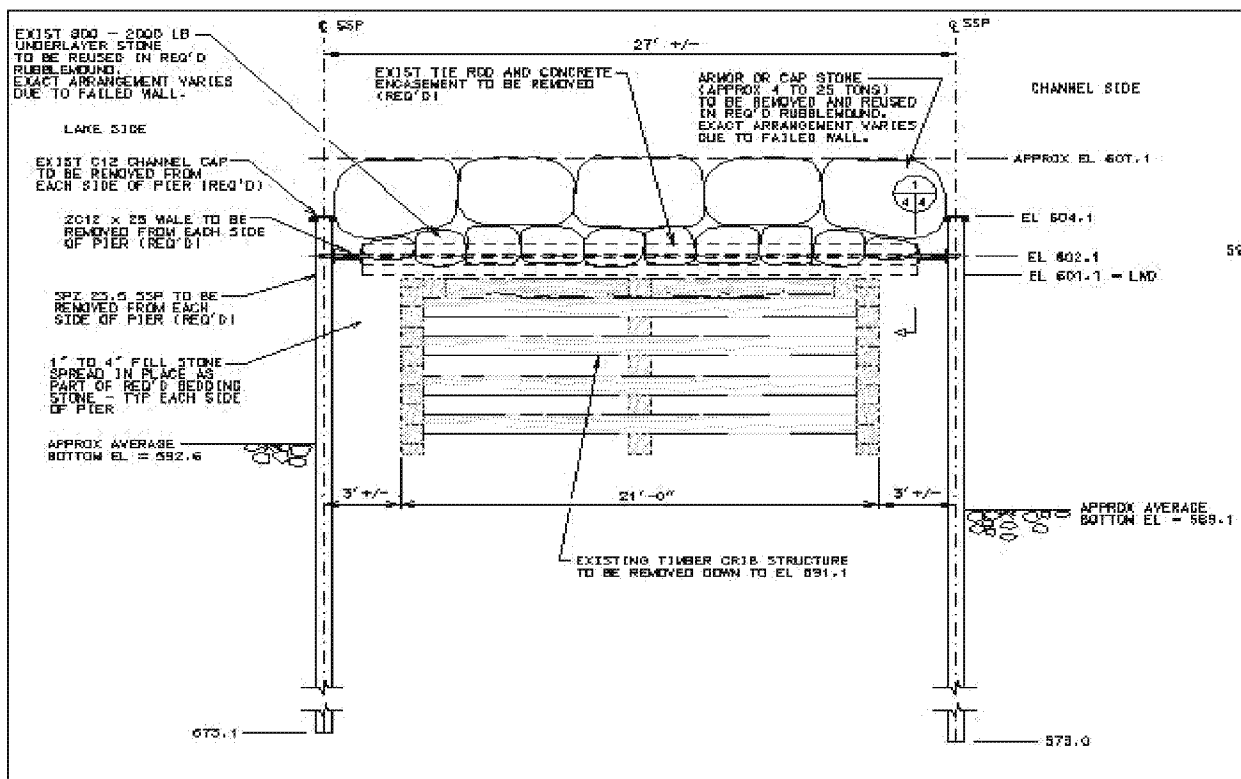
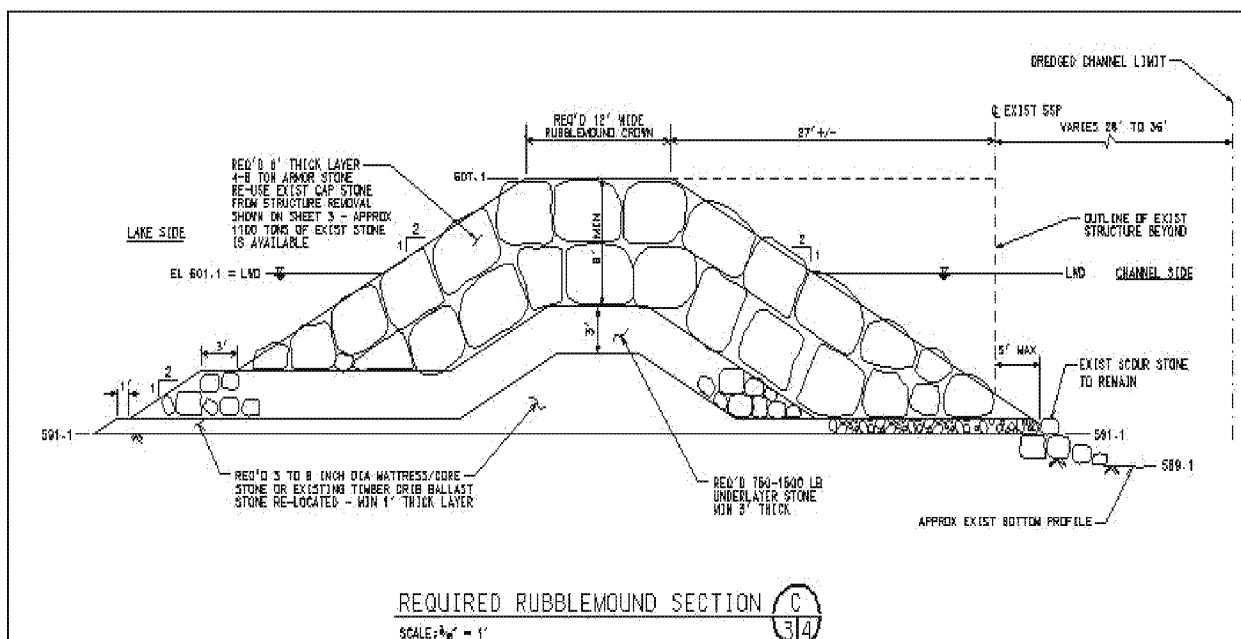Figure K-7a. Timber crib design details referenced to IGLD85 (601.1) Datum.

Figure K-7b. Rubblemound design details referenced to IGLD85 (601.1) Datum.

f. Geodetic and water level references and uncertainties. Based on the published data, the geodetic and "tidal datum" relationships at Tidal Bench Mark "D-135" could be tabulated as shown in Table K-2.

Table K-2. Elevations at Tidal Bench Mark "D-135."

Datum	Elevation	Referenced From	Estimated Uncertainty	Relative to
NGVD29	613.22 ft	VERTCON transform	±0.3 ft	NSRS
IGLD85	613.01 ft	Tidal BM D-135	±0.2 ft	NGS
NAVD88	613.11 ft	NSRS	±0.1 ft	NSRS
Dynamic Ht	613.21 ft	Tidal BM D-135	±0.2 ft	NWLON

K-4. Background on Establishment of IGLD85. The following paragraphs provide additional background on the establishment of IGLD85 in the Great Lakes. They are excerpted from "*Establishment of International Great Lakes Datum (1985)*" (IJC 1995) by The Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data.

The establishment of the International Great Lakes Datum (1955), or IGLD (1955), was one of the first major accomplishments of the Coordinating Committee. Accordingly the reference zero point was established at Point-au-Père, Quebec and first-order leveling begun in 1953 was completed in 1961. The established bench mark elevations were published In September 1961 (A second edition was also published with some revisions In December 1979). The result of this effort was the International Great Lakes Datum 1955. This datum was implemented January 1, 1962, and used for the following 30 years, until the effects of crustal movement, the development of a common datum between Canada, the United States, and Mexico, new surveying methods, and the deterioration of the zero reference point gauge location made it desirable to revise the datum. The Vertical Control-Water Levels Subcommittee undertook the revision of IGLD (1955) beginning In 1976 and this effort has resulted in International Great Lakes Datum (1985) ... The development of the NAVD (1988) ... was to Include vertical control networks of the U.S., Canada and Mexico, as well as International Great Lakes Datum data. For NAVD (1988), a minimum-constraint adjustment was performed also holding fixed the primary bench mark at Pointe-au-Père/Rimouski, Therefore, IGLD (1985) and NAVD (1988) are one and the same. The only difference between IGLD (1985) and NAVD (1988) is that the IGLD (1985) bench mark elevations are published as dynamic heights and the NAVD (1988)

elevations are published as Helmert orthometric heights Geopotential numbers for individual bench marks are the same in both height systems.

Dynamic height values. The surveying and mapping community uses several different heights systems. Two systems, orthometric and dynamic heights, are relevant to the establishment of IGLD (1985) and NAVD (1988). The geopotential numbers for individual bench marks are the same in both height systems. The requirement in the Great Lakes basin to provide an accurate measurement of potential hydraulic head is the primary reason for adopting dynamic heights, it should be noted that dynamic heights are basically geopotential numbers scaled by a constant of 980.6199 gals, normal gravity at sea level at 45 degrees latitude. Therefore, dynamic heights are also an estimate of the hydraulic head. Consequently points that have the same geopotential number have the same dynamic height. Following are some of the advantages of dynamic heights:

(1) In crustal movement studies, differences in the dynamic elevation of bench marks from lake to lake can be compared regardless of the route along which the leveling is done. This is also possible in the orthometric height system and with geopotential numbers.

(2) Difference in dynamic heights and in geopotential numbers give an accurate measure of the potential hydraulic head between selected points. This is not true of orthometric heights.

Hydraulic Corrector. The water surfaces of the Great Lakes are considered to be geopotentially equal. Therefore, on any particular lake, at the time a new vertical datum is established, all Mean Water Level (MWL) values for gauging stations around the lake should coincide. The MWL is the average water surface for the summer months (June - September) for the years 1982-1988 referenced to the gauging station Primary Bench Mark dynamic height ... the MWL at each gauging station was treated as a bench mark In the network adjustment. Following the adjustment ..., the MWL values at each gauging station on a lake were slightly different. The differences are due to cumulative differences in the leveling adjustments. The Committee decided to apply a Hydraulic Corrector so each gauge on a lake has the same MWL as the Master Station for the lake. This is accomplished by holding the Master Station as the controlling value and comparing all other gauging stations to it. The Master Stations for each lake are:

Lake Ontario	*Oswego, New York*
Lake Erie	*Fairport, Ohio*
Lake St. Clair	*St. Clair Shores, Michigan*
Lake Huron	*Harbor Beach, Michigan*
Lake Michigan	*Harbor Beach, Michigan*
Lake Superior	*Marquette, Michigan*

The Hydraulic Corrector (HC) was obtained by subtracting the MWL at the Master Station (MWL_{Master}) from the MWL at the subordinate gauging station in question (MWL_{Sub}). The answer retains its arithmetic sign. The Hydraulic Corrector may be positive or negative and is subtracted algebraically.

K-9

$$HC = MWL_{Sub} - MWL_{Master}$$

where:

> HC = *Hydraulic Corrector for subordinate gauge.*
>
> MWL_{Sub} = *Mean Water Level at Subordinate Gauging Station on a lake for the summer months (June - September) of 1982 - 1988. The MWL is referenced to the Subordinate Gauging Station Primary Bench Mark Dynamic Height.*
>
> MWL_{Master} = *Mean Water Level at Lake Master Station for the summer M W' months (June - September) of 1982 - 1988. The MWL is referenced to the Master Station Primary Bench Mark Dynamic Height.*

The water surface elevation ($WS_{IGLD1985}$) is obtained by subtracting the Hydraulic Corrector (HC) from the Dynamic Water Surface Elevation ($WS_{Dynamic}$).

$$WSI_{GlD1985} = WS_{Dynamic} - HC$$

where:

> $WS_{IGLD1985}$ = *Published Water Surface Elevation on IGLD (1985) for a selected gauging station. The value may be an instantaneous value, or a daily, monthly, or annual mean.*
>
> $WS_{Dynamic}$ = *Water Surface elevation referenced to Dynamic Height.*
>
> HC = *Hydraulic Corrector for a selected gauging station. The value may be positive or negative*

The Hydraulic Corrector at each gauge site on the lake has been incorporated into the data retrieval and storage process. As such, water level information stored at the site mechanically or electronically, at the National Oceanic and Atmospheric Administration (NOAA) or the Department of Fisheries and Oceans (DFO) computers, or in printed form, are in IGLD (1985) and do not require any further adjustment

The advantages of IGLD (1985), leading to the Coordinating Committee recommendations, may be summarized as follows:

> *(1) Elevations, consistent with one another as of a recent date (1985), are provided for bench marks throughout the Great Lakes-St, Lawrence River system, with the reference zero at Pointe-au- Père/Rimouski.*

(2) The elevations given on this datum are based on the dynamic principle, and are therefore more suitable for hydraulic studies. Elevations on this new datum will greatly facilitate hydraulic, hydrographic and other engineering studies.

K-5. Tabulation of Great Lakes and Connecting Channels Water Level Datums (NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER
ST. LAWRENCE RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
8311030	73.88	242.4	Ogdensburg, NY St. Lawrence River
8311062	74.07	243.0	Alexandria Bay, NY St. Lawrence River
LAKE ONTARIO (CHART DATUM/LWD 74.2 M - 243.3 FT.)			
9052000	74.2	243.3	Cape Vincent, NY Lake Ontario
9052030	74.2	243.3	Oswego, NY Lake Ontario
9052058	74.2	243.3	Rochester, NY Lake Ontario NY
9052076	74.2	243.3	Olcott, NY Lake Ontario
NIAGARA RIVER (NON NAVIGABLE WATERS)			
9063007	N/A	N/A	Ashland Ave., NY Niagara Falls-Below the falls
9063009	N/A	N/A	American Falls, NY Niagara Falls-Above the Falls
9063012	N/A	N/A	Niagara Intake, NY Niagara River – Power diversion water intakes
LAKE ERIE (CHART DATUM/LWD 173.5 M - 569.2 FT.)			
9063020	173.5	569.2	Buffalo, NY Lake Erie
9063028	173.5	569.2	Sturgeon Point, NY Lake Erie
9063038	173.5	569.2	Erie, PA Lake Erie
9063053	173.5	569.2	Fairport, OH Lake Erie
9063063	173.5	569.2	Cleveland, OH Lake Erie
9063079	173.5	569.2	Marblehead, OH Lake Erie
9063085	173.5	569.2	Toledo, OH Lake Erie
9063090	173.5	569.2	Fermi Power Plant, MI Lake Erie
DETROIT RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
9044020	173.58	569.5	Gibraltar, MI Detroit River
9044030	173.95	570.7	Wyandotte, MI Detroit River
9044036	174.08	571.1	Fort Wayne, MI Detroit River
9044049	174.34	572.0	Windmill Point, MI Detroit River

K-5 (Continued). Tabulation of Great Lakes and Connecting Channels Water Level Datums
(NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER

LAKE ST. CLAIR (CHART DATUM/LWD 174.4 M - 572.3 FT.)

9034052	174.4	572.3	St. Clair Shores, MI Lake St. Clair

ST. CLAIR RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)

9014070	174.58	572.8	Algonac, MI St. Clair River
9014080	175.08	574.4	St. Clair State Police, MI, St. Clair River
9014084	175.35	575.3	Marysville, MI St. Clair River (in-active)
9014087	175.50	575.8	Dry Dock, MI St. Clair River
9014096	175.77	576.7	Dunn Paper, MI St. Clair River
9014098	175.93	577.2	Fort Gratiot, MI St. Clair River

LAKE HURON (CHART DATUM/LWD 176.0 M - 577.5 FT.)

9075002	176.0	577.5	Lakeport, MI Lake Huron
9075014	176.0	577.5	Harbor Beach, MI Lake Huron
9075035	176.0	577.5	Essexville, MI Lake Huron
9075059	176.0	577.5	Harrisville, MI Lake Huron
9075065	176.0	577.5	Alpena, MI Lake Huron
9075080	176.0	577.5	Mackinaw City, MI Lake Huron
9075099	176.0	577.5	De Tour Village, MI Lake Huron

LOWER - ST. MARY'S RIVER (CHART DATUM/LWD-SEE STATION VALUE FOR SLOPING SURFACE)

9076024	176.03	577.5	Rock Cut, MI St. Mary's River
9076024	176.12	577.8	West Neebish Island, MI
9076028	176.12	577.8	Lookout Station #4, MI St Mary's River
9076032	176.29	578.4	Little Rapids, MI St Mary's River
9076060	176.38	578.7	U.S. Slip, MI St. Mary's River

UPPER - ST. MARY'S RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)

9076070	183.00	600.4	S.W. Pier, MI St. Mary's River

K-5 (Concluded). Tabulation of Great Lakes and Connecting Channels Water Level Datums (NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER
LAKE MICHIGAN (CHART DATUM/LWD 176.0 M - 577.5 FT.)			
9087023	176.0	577.5	Ludington, MI Lake Michigan
9087031	176.0	577.5	Holland, MI Lake Michigan
9087044	176.0	577.5	Calumet Harbor, IL Lake Michigan
9087057	176.0	577.5	Milwaukee, WI Lake Michigan
9087068	176.0	577.5	Kewaunee, WI Lake Michigan
9087072	176.0	577.5	Sturgeon Bay Canal, WI Lake Michigan
9087079	176.0	577.5	Green Bay, WI Lake Michigan
9087088	176.0	577.5	Menominee, MI Lake Michigan
9087096	176.0	577.5	Port Inland, MI Lake Michigan
LAKE SUPERIOR (CHART DATUM/LWD 183.2 M - 601.1 FT.)			
9099004	183.2	601.1	Point Iroquois, MI Lake Superior
9099018	183.2	601.1	Marquette C.G. MI Lake Superior
9099044	183.2	601.1	Ontonagon, MI Lake Superior
9099064	183.2	601.1	Duluth, MN Lake Superior
9099090	183.2	601.1	Grand Marais, MN Lake Superior

APPENDIX L

Computing Historical Subsidence Rates in Southeast Louisiana from USACE Gage Data

L-1. Purpose. This appendix describes procedures for evaluating subsidence rates based on long-term gage records. It is extracted from internal studies performed in the USACE New Orleans District.

L-2. Abstract. The New Orleans District records stream and tide stages at numerous gaging sites throughout its district. Many of these stage data sets extend as far back as 1940. The data through 1998 have been published by the District in "Stage and Discharge" books and much of it has been converted to digital format. As such, it is often readily accessible, reasonably well documented, and may provide an independent means to investigate and determine reliable rates of local subsidence and/or validate rates determined via geodetic survey analysis.

L-3. Introduction. Subsidence, or the generally downward motion of the earth's surface with respect to some "fixed" vertical datum (e.g., NAVD88 or a particular Mean Sea Level epoch), is perceived as a significant threat to coastal Louisiana. It has been detected in both geodetic leveling data throughout the region and at several NOAA tide gages along the Louisiana coast (Shinkle and Dokka, 2004; Dokka, 2006)[1]. From the referenced studies, it appears that rates of subsidence in the region are spatio-temporally variant and often significantly greater than even the highest estimated rates of eustatic sea-level rise. Consideration of the forces and conditions that cause subsidence - and the proportion that each force or condition contributes to total subsidence at a given point and at a given time (Gonzalez and Tornqvist, 2006; Dokka, 2006) - is beyond the scope of this work. The sole purpose here is to extract reliable historical rates of subsidence at various USACE gages and, when/if possible, determine the relationships among the various vertical geodetic datum/epochs and local mean-sea-level epochs.

L-4. Data. Five USACE tide gage stations in the New Orleans area were selected for this study. They are as follows: 76040 - The Intracoastal Waterway (IWW) at the Paris Road Bridge; 76060 - The Inner Harbor Navigation Canal (IHNC) at the Seabrook Bridge; 76120 - The IHNC at the Florida Avenue Bridge; 85675 - Lake Pontchartrain at Irish Bayou; 85700 – Lake Pontchartrain at the Rigolets (see Figures L-1, L-2, and L-3).

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[1] References cited in this Appendix are listed in Section L-10.
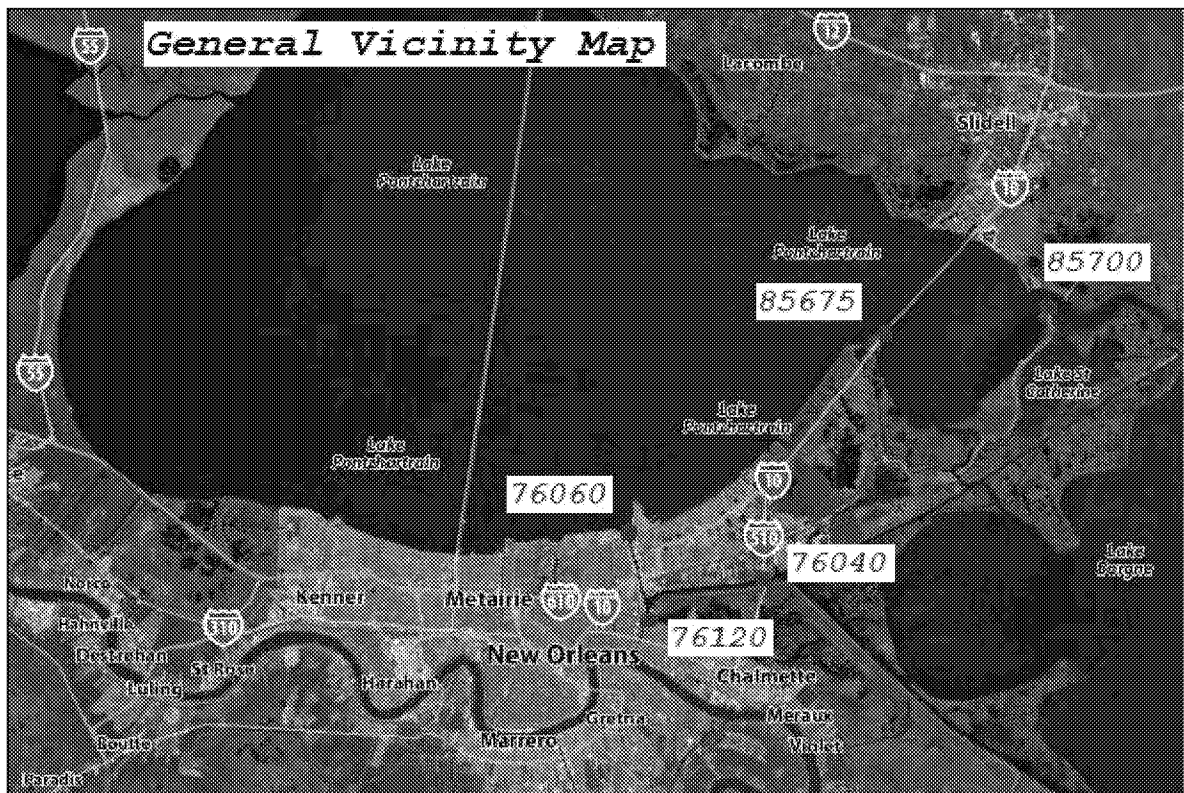
Figure L-1. Vicinity map of Greater New Orleans showing gage locations.

    a. These tidal gages were selected because of their proximity and relevance to post-Katrina reconstruction activities in New Orleans East, the availability of near-continuous, long-term (40+ years) digital data, and adequate documentation of periodic gage inspections and adjustments. This collection of gages also encompasses an area of detailed study on modern-day tectonic subsidence in coastal Louisiana performed by Dokka (2006).

Figure L-2. Vicinity map of East New Orleans showing gage locations.
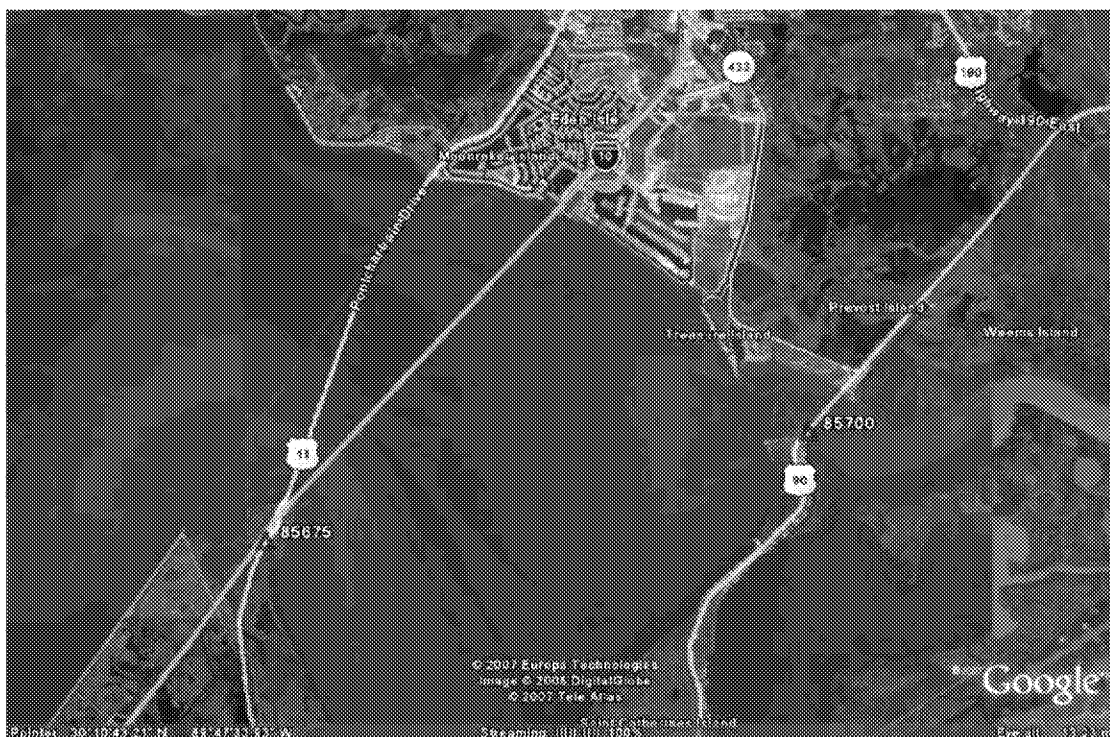
Figure L-3. Vicinity map of Western Lake Pontchartrain showing gage locations.

b. The available digital data ranges for the selected gages are shown in Table L-1. The data sets are, for the most part, continuous through their respective time periods, but there are several gaps (of days, weeks, and sometimes months) in each of the data sets. More significantly, all of the original data sets are nominally referred to as "Daily 8 A.M. Stage Readings in Feet." Occasionally, stage readings were made directly by an observer at times other than 8 A.M. In addition, the type of gage employed, its precise location, and the type of structure on which it was mounted were all altered one or more times at each of the gaging sites over the life of its operation. As such, the data sets may be somewhat "noisy," as compared to a theoretically continuous data set of hourly or six-minute-interval stage readings from a single, calibrated instrument at a fixed location.

Table L-1. The Available Digital Data Ranges for Selected Gages

Gage	Year Period Began	Year Period Ended
76040	1959	2007
76060	1962	2005
76120	1944	2003
85675	1959	2000
85700	1961	2001

c. The most significant artifacts evident in each of the data sets were the discontinuities resulting from the deliberate change in the vertical position of the gage zero with respect to nearby benchmarks. These alterations were periodically carried out so that the "zero" of the gage would correspond to the "zero" of a particular epoch of a vertical geodetic datum (see Figure L-4, for example). Fortunately, these vertical movements or adjustments of the gage zeros are reasonably well documented in the gage inspection records together with the explanatory notes in the "Stages and Discharges" books. Accounting for these adjustments was essential to the development of continuous, normalized data sets of daily, 8 A.M. stage readings from which monthly means could be reliably computed and subsidence rates determined.
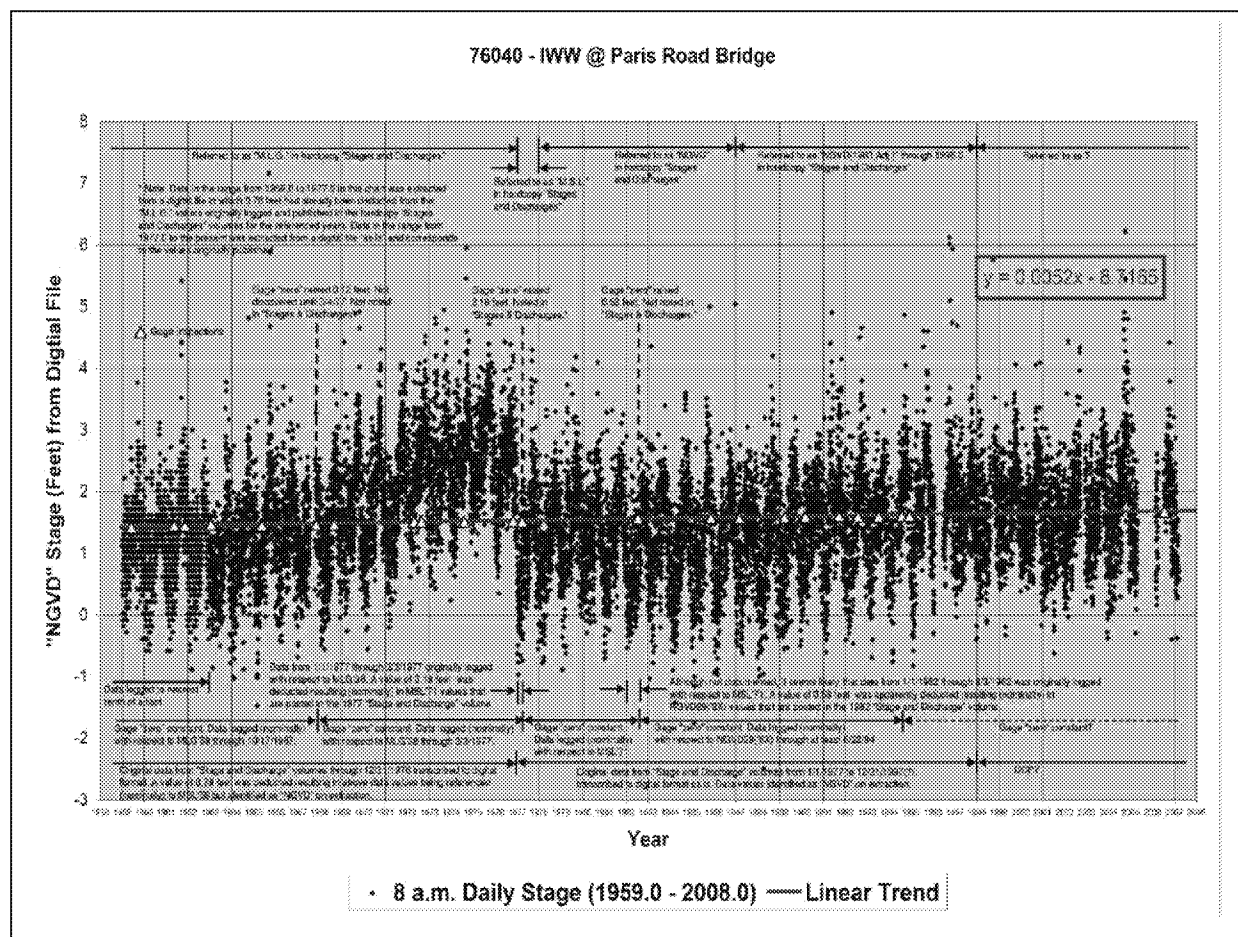
Figure L-4. Gage readings for gage 76040 – IWW @ Paris Road.

L-5. Data Normalization. In normalizing the digital stage data, it was first necessary to account for differences between the digital stage values extracted from the USACE database, and those corresponding values recorded in the "Stages and Discharges" books. These differences are due to a bulk shift - applied on extraction by USACE software - to that portion of a given stage data set originally recorded with respect to a gage zero intentionally offset from nominal mean-sea-level (i.e., "Mean-Low-Gulf" or "-10.00 ft MSL" or "-20.55 ft MSL). This is apparently done in order to roughly harmonize it with subsequent stage values nominally referenced to MSL/NGVD. Removal of this shift was necessary to reproduce the actual stage values recorded

L-5

in the "Stage and Discharge" books. Following this adjustment, the digital stage data were corrected for intentional vertical movement of the gage zero due to epoch updates as indicated in the inspection records. The desire here was to compute, as nearly as possible, the stage data set that would have been generated if the gage zero had never been intentionally moved from its initial vertical position with respect to the surrounding terrain and associated benchmarks. Applying the corrections for gage zero movement noted in Figure L-4 resulted in the "normalized" data set shown in Figure L-5.
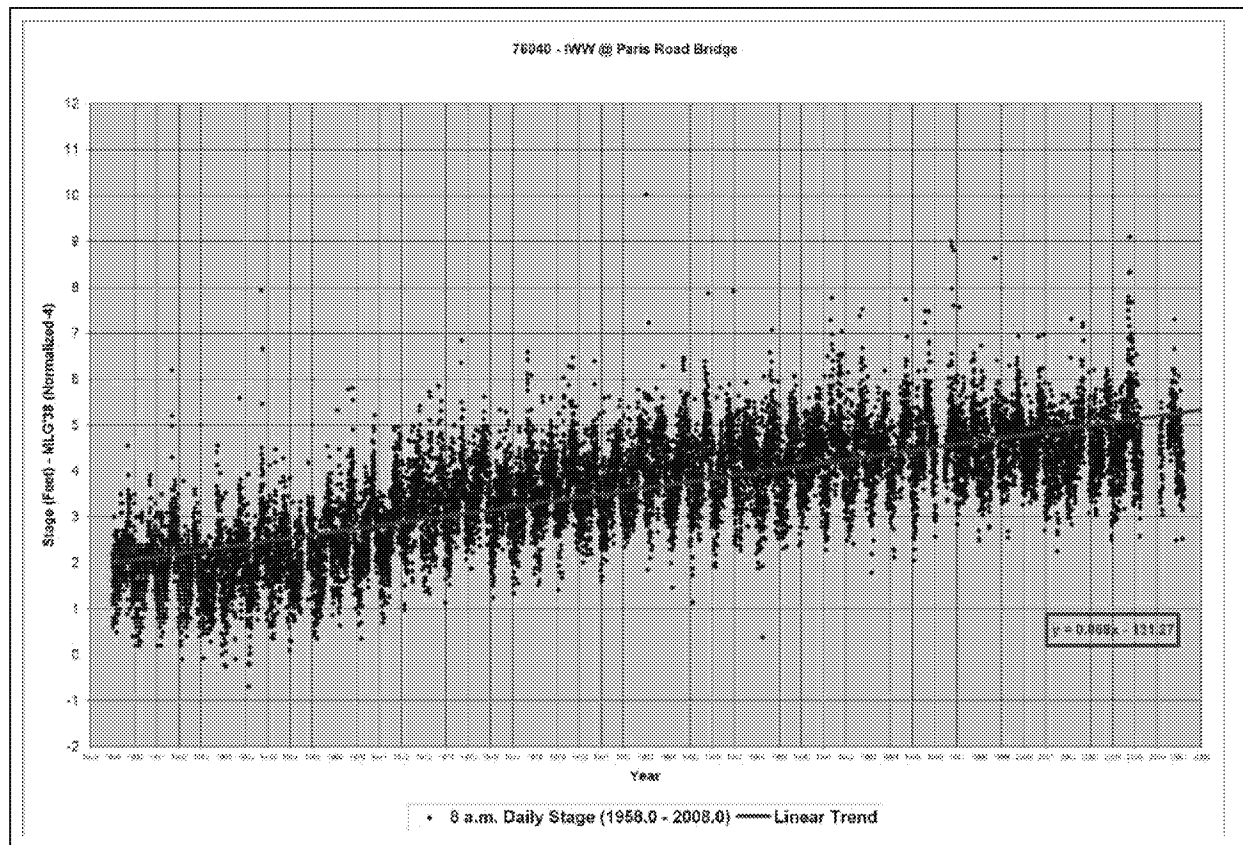
Figure L-5. Normalized gage readings for gage 76040 – IWW @ Paris Road.

a. The review, analysis, and processing steps undertaken with gage 76040, as indicated above in Figures L-4 and L-5, were similarly carried out with respect to the remaining four gages. As a final quality assurance step, all five normalized data sets were differenced against one another (see Figures L-6 through L-15). If all intentional gage zero movement has been accounted for (and if no accidental zero movement or loss of calibration has occurred), then one would expect the graph of the differences to be relatively consistent, smooth and continuous. If subsidence rates were generally the same at any two gages under consideration, the graph would be essentially flat as well. Where rates differ, one would expect that the magnitude of that rate difference would be born out in the slope of the graph.
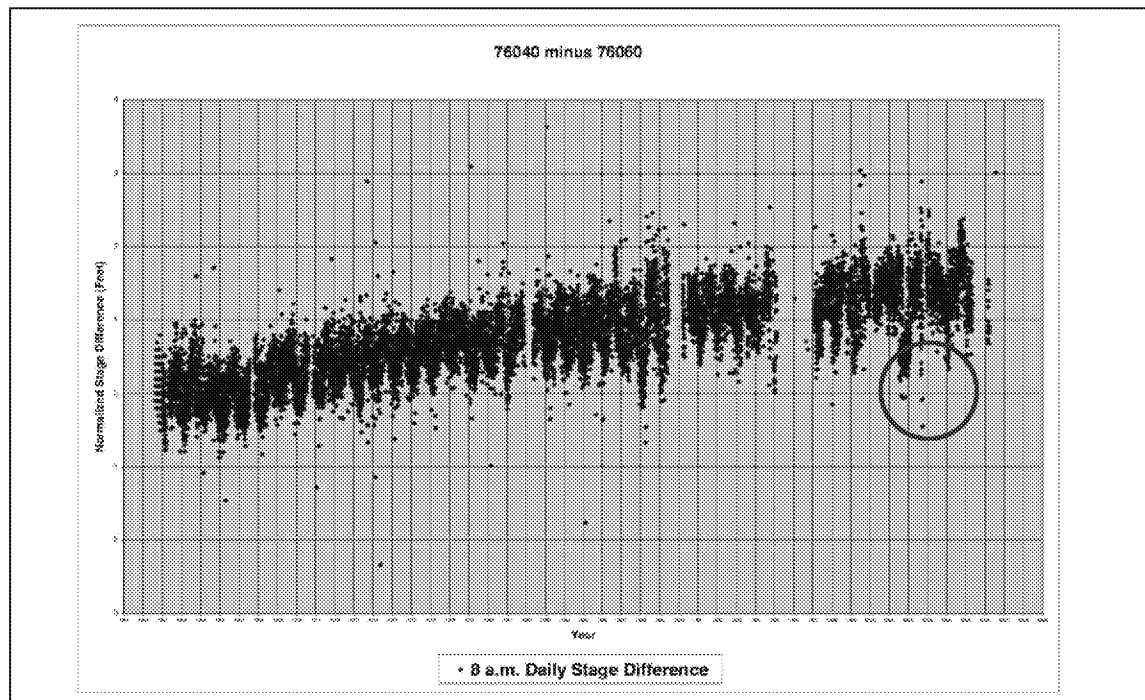
L-6

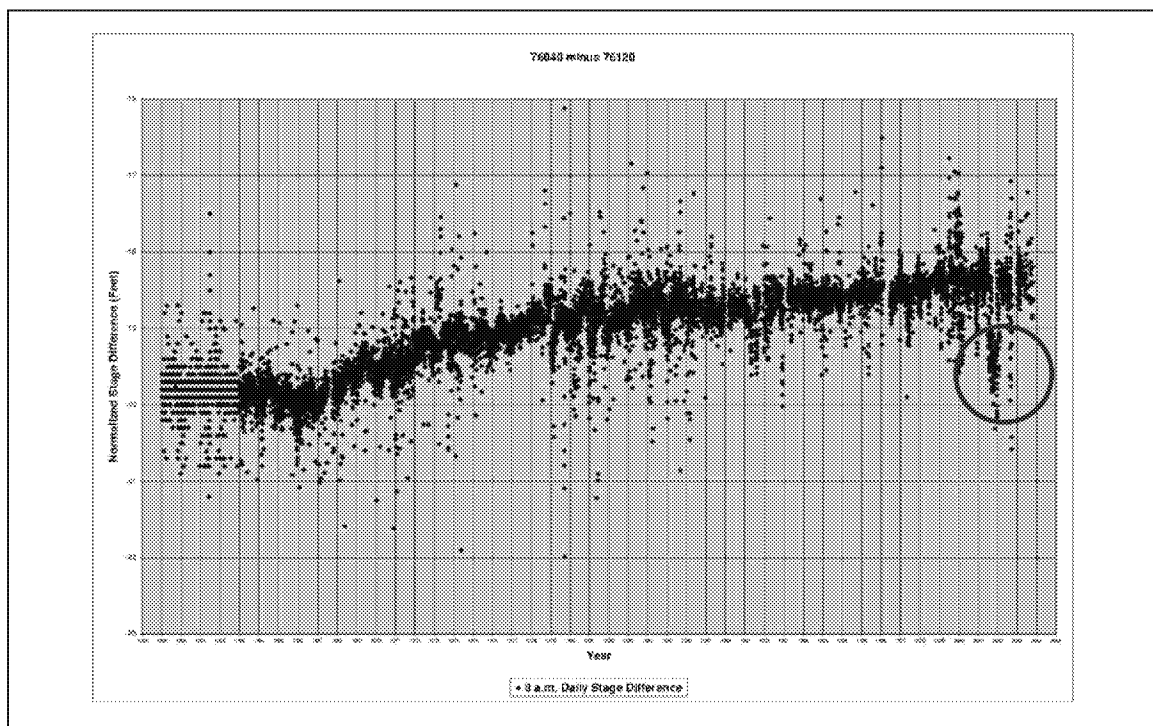Figure L-6. Normalized gage readings for Gage 76040 – Gage 76060.

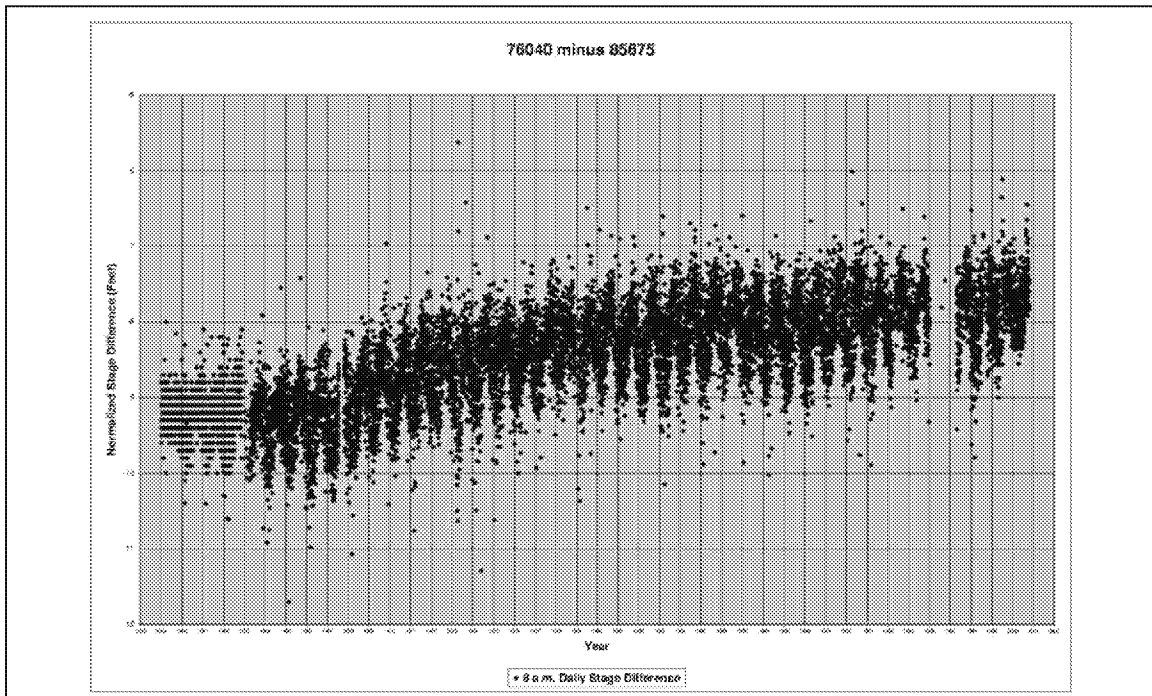Figure L-7. Normalized gage readings for Gage 76040 – Gage 76120.

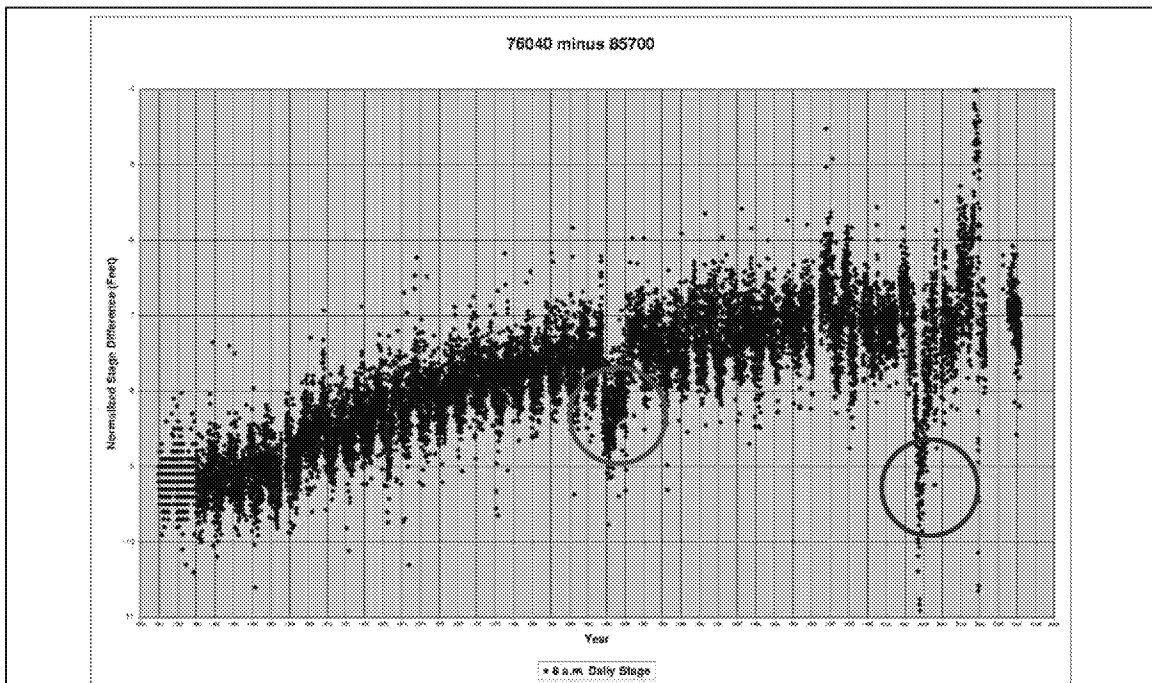Figure L-8. Normalized gage readings for Gage 76040 – Gage 85675.

Figure L-9. Normalized gage readings for Gage 76040 – Gage 85700.